Concrete Model Code for Asia - Maintenance

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Summary

This paper is largely based on the draft report of the Working Group on Repair and Maintenance of concrete structures of the International Committee of the Japan Concrete Institute. It tries to present a comprehensive methodology for the maintenance of concrete structures during their service life, as may be necessitated by exposure to natural environment and ensuing deterioration. The treatment of the subject has been kept general and open ended in order that the committee report can be used as a basis for developing specifications and manuals for structures using different types of concrete, made in different environmental conditions and expected to perform diverse functions.

1. Introduction

In recent years, advances in construction methods have made it possible for concrete structures to be built in severe environmental conditions. At the same time the cost of unplanned and piece-meal remedial action have risen. Further, the need to extend the service life of existing structures and a better understanding of the deterioration mechanisms in concrete, has led to efforts to develop a rational methodology for the maintenance of concrete structures.

The goal for maintenance of a concrete structure during its service life is to ensure that its performance meets a predetermined criteria. It is therefore, important that effort be made to quantitatively define parameters which could be used to monitor the extent of deterioration and lay down minimum required performance levels. However, since concrete is used in different structures (buildings, dams, bridges, etc.), which perform under different environmental and operational conditions, it is not possible to lay down identical performance criterion for all (concrete) structures.

Thus, in this paper, and indeed in the Committee Report, the subject of maintenance of concrete structures has been treated only in a general manner, so that the document can be used as a basis for developing specific quantitative parameters, specifications and manuals for different structures - using different types of concrete, made in different environmental conditions and expected to perform diverse functions - on a case-to-case basis. Indeed, different organisations charged with the responsibility of maintaining concrete structures like the railways, etc. have developed their own tools an know-how for their specific needs. The effort in the committee report and indeed in this paper is to present a basic frame-work, which can be used to develop maintenance strategies in a large cross-section of concrete structures. Such a framework, will help evolve common strategies in addressing issue of mutual interest. Now, an overall maintenance strategy should comprehensively encompass, a rational basis for the maintenance, inspection, estimation of deterioration level and rates, evaluation of structural integrity, and remedial actions, as may be required. These aspects have been briefly discussed in the following paragraphs.

2. Basis of maintenance

On the basis of factors such as, importance of the structure, design service life, impact on a third party, environmental conditions, cost involved and ease of maintenance, maintenance action could be classified into different categories. For example, critical structures (e.g. dams, nuclear power plants), structures required to have a long service life (e.g. monuments), or those situated in very harsh environments (e.g. marine), may be classified to fall in a higher priority category maintenance, than for example, a multi-storey building. Similarly, criteria for classifying structures into other levels of priority need to be developed. It should be pointed out that certain structures, where any maintenance action is very difficult to execute (e.g. underwater), may have to be classified separately.

3. Inspection

Occurrence of deterioration and/or change in its performance in a structure is detected through inspection. Obviously if undesirable signs of deterioration can be detected early, suitable timely remedial action can be initiated. Actual locations for inspection, items recorded and tools used, should be carefully selected so that the desired information can be obtained accurately.

3.1 Types of inspection

Beginning with initial inspection carried out immediately after completion of construction, (or even repair or strengthening work), the structure needs to be periodically inspected. The objective of the initial inspection is essentially to compile the work records, record any deviations from the design / drawings, establish the initial state of the structure (before being put into operation), and prepare final documents, which can serve as basis for further maintenance action.

Now, while the structure is in service, routine and regular inspection need to be carried out to determine whether or not detailed inspection is required. The frequency and rigour of such inspections may be determined depending upon factors such as likely mechanism of deterioration, environmental conditions, importance of the structure, and classification of maintenance action. The underlying assumption is that a decision on whether or not to initiate remedial action, must be based on data gathered during a detailed inspection.

In addition to routine, regular or detailed inspections, as outlined above, extraordinary inspection may also be carried out to assess the extent of damage and need for remedial action, after a structure has been subjected to an accidental load, such as, earthquake, storm, flood, fire, collision with a vehicle or ship.

Further, whereas inspections could at best provide data at a particular point in time, the need to (continuously) monitor deterioration and/or performance of critical structures, through continuous recording of appropriate data, should not be lost sight of. In such cases, appropriate sensors and recording devices should be fixed to the structure, so that relevant data can be collected at any time.

3.2 Equipment used for inspection

Visual inspection : Visual inspection could provide vital information about the changes in the performance and / or deterioration in terms of appearance of pop-outs, cracks, stains, etc. Visual inspection, therefore, needs to be carried out systematically to obtain and record relevant information. It may be appropriate to treat visual inspection as an integral part of periodic inspection, and the frequency and rigour of routine / regular inspections may be adjusted according to the results of visual inspection. Photographic records could also serve as a source for monitoring changes in a structure over its service life, though the information in most cases may be limited to changes at the surface.

Tools during inspection : A detailed or comprehensive treatment of the methods available for non-destructive testing methods is outside the purview of this report. However, it is only appropriate to mention that most of the tools used for non-destructive testing and evaluations of structures have inherent limitations and a range of conditions over which the results are reliable. These should be borne in mind when choosing the tools to use and/or interpreting the data obtained. For example, though a rebound hammer test is often used to estimation of strength of concrete, it actually measure only the surface hardness and is susceptible to variation on account of factors such as roughness, wetness, and properties and proportion of aggregates in the mix. Though correction factors for some of these factors are often given in literature, the fact that the actual parameter recorded is hardness should not be lost sight of. Similarly, the readings for the natural potential of the reinforcing bars, often used in cases of assessing reinforcement corrosion, are highly dependent on the level of saturation of the cover concrete.

3.3 Locations for inspection

Critical locations in the structure for inspection need to be identified. The choice of these locations should be governed not only by structural but also environmental (which directly affects deterioration) considerations. As discussed in greater detail later on, these two approaches could help obtain (almost) independent assessments of the structure. The number of location identified for the purpose could be determined by considerations such as the importance of the structure, nature of structural and environmental loads, tools of inspection, and resources available.

4. Estimation of deterioration levels and rates

As mentioned above, appropriate deterioration parameters need to be identified depending upon the likely mechanism of deterioration, etc. Now, though inspection procedures used may provide the desired information about the instantaneous value of such parameters, appropriate models are needed to be able to estimate rates of change in the deterioration parameters. These models should take into account factors like the likely mechanism and observed characteristics of deterioration, and using the information available from construction records, previous inspections, etc.

For example in the case in chloride induced reinforcement corrosion, chloride concentration at a certain depth from the surface or natural potential of the reinforcing bars could be deterioration parameters. Further, proper sampling and analysis may yield the chloride concentration in concrete at a certain depth from the surface, but appropriate diffusion (or any other) models are needed to estimate the rate chloride ingress into concrete. This is required in order to estimate other parameters, such as, time that may elapse before concentration at a given location (say neighbourhood of the bars to reach a critical level (for example, the critical level required to render ineffective the passivating film around the bars). Relevant information about values of the apparent coefficient of diffusion and the surface chloride concentration should be based on the properties and proportions of the materials (especially cement) used, environmental conditions, etc.

5. Evaluation of results and structural integrity

Periodic (routine and/or regular) inspection should provide a basis for detailed inspection, which in turn should provide the input to the decision on whether or not to initiate remedial action. Though the issue of remedial action has been dealt with in some detail separately at a later stage, the importance of proper assessment of integrity of the structure in terms of performance parameters and the level of deterioration in terms of deterioration parameters, is discussed here.

5.1 Integrity of a structure

This could be taken to refer to the ability of a structure to perform its design functions (the deterioration, notwithstanding). In terms of structural performance, this could be measured in terms of the load carrying capacity, deflections, etc. In certain other cases, where the appearance of the structure is of importance, discoloration and/or staining could be of concern. Thus, from an operational and functional point of view, it is necessary to establish a minimum or threshold level of performance level in terms of relevant parameters, depending upon the environmental conditions and type, importance and maintenance classification of the structure.

5.2 Deterioration level

This can be taken to be the comprehensive assessment of the deterioration in a structure, made on the basis of results from inspection(s), study of design / construction records, models for estimation of future rate of deterioration, importance and maintenance classification. This can be arrived at only after considering the input from the various deterioration parameters, as may be available. For example, an overall assessment of deterioration due to chloride induced reinforcement corrosion can be based only after parameters such as chloride concentration levels, natural potential of reinforcing bars, extent of longitudinal cracks, appearance of rust stains, etc. are all taken into account.

5.3 Decision making

Based on the input from two essentially independent assessments - the critical performance level and the deterioration level - as described above, a decision needs to be taken to initiate detailed inspection and/or appropriate remedial action. However, it is at times difficult to be able to directly link the level of deterioration to the changes in the structural integrity, and in that case, fixing a maximum level of deterioration (in addition to the minimum level of performance) may have to be resorted to.

For example, if changes in structural integrity on account of reinforcement corrosion are considered, substantial and unacceptable levels of changes in structural behaviour have been reported even at fairly low levels of corrosion (in terms of loss in weight of the bars). In such a case, it is important that till such time as the mechanism of corrosion is better understood, a maximum 'acceptable' level of chloride concentration in concrete, or an unacceptable level of natural potential of the reinforcement may be adopted, though their relevance, per se, to the structural performance is limited.

6. Remedial action

On the basis of input from inspection - in terms of changes in performance levels, deterioration level, threat to the environment, etc. - suitable remedial measures may need to be initiated, as briefly discussed below. A complete plan should be drawn up for the works, which should be carried out with minimum disturbance to the environment.

6.1 Repair

Refers to action taken to prevent or slow down further deterioration in a structure and/or reduce the possibility of damage to the environment or any third party. Repair may be undertaken when there is no serious change in structural integrity and action may be limited to surface applications, sealing of cracks, etc.

6.2 Strengthening

Refers to action taken to restore or improve its load bearing capacity to at least the design level. Strengthening works should be preceded by a thorough investigation, including remaining design or desired service life, likely mechanism, causes and extent of deterioration, remaining and desired load bearing capacity, importance of the structure, maintenance classification and previous remedial action taken.

It may also be noted that within the framework of maintenance of a structure, the possibility of having to strengthen it even outside the considerations of deterioration and durability cannot be ruled out, e.g. in the case of adoption of more stringent design criteria and/or (an upward) revision of loads. Thus, it is only appropriate that when a structure is strengthened, efforts are made to go through the entire design procedure and ensure compliance with prevailing requirements.

Further, strengthening may require use of materials and methods, which were not used in the original construction and therefore specifications and other tools need to be developed for proper quality control. In fact, a proper plan of action taking into account all aspects of the job should be drawn up.

6.3 Other remedial actions

Action such as more intensified inspection (increasing the frequency and/or rigour of inspection), usage restriction (speed or load limit), landscaping (application of surface paints), dismantling and removal, etc. can also be called remedial action. When a deteriorated structure poses an immediate threat to the environment, user or any third party, suitable emergency action should be taken immediately, while the plans for further action are drawn up.

7. Maintenance of records

Complete records including details of design, construction, inspection and evaluation procedures, plans and execution of any repair and/or strengthening work undertaken etc., need to be made and retained in a manner that the information is easily accessible throughout the service life of a structure. It may be noted that even when these records may not be required for the maintenance of a certain structure, they may provide invaluable information for the design, construction and maintenance of other (similar) structures. Also, these records may provide relevant information about the structure as a whole or its individual members and preserved for while the structure is in service.

8. Concluding remarks

Concrete is not a maintenance-free material and concrete structures require careful maintenance - encompassing inspection and remedial action, to ensure that they continue to discharge their design functions throughout the service life. Details of inspection procedures, interpretation of results and remedial action have been deliberately left out of the discussion here and an effort has been made to only assemble a broad framework, which can be used for a large cross-section of concrete structures.

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Load-Rating Based Bridge Deterioration Model

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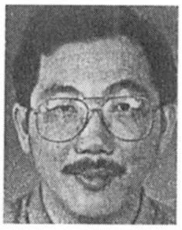

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Summary

In managing in-service bridges it is very important that the change in bridge performance over time be known. Existing bridge management systems (BMS), for example, Pontis [1] uses the relationship between condition rating and time. However, load rating is also a useful measure of performance. It is logical that the relationship of the load rating and time should also be useful in BMS applications. The authors had proposed a method based on survival analysis to study such relationship. The deterioration process is modeled as a semi-Markov process.

1. Introduction

Bridge deterioration models are an essential component of a computerized bridge management system (BMS). It allows prediction of future bridge performance and needs. Existing BMSs, for example, Pontis [1] model bridge deterioration as the decline of condition rating over time. Condition rating is an ordinal scale consisting of numerical values assigned to represent each condition state of the bridge or its components. If we define state 1 as the best state, state 2 as the next state, so on and so forth then a typical bridge deterioration model would look like a stepped function as in Fig. 1.

Condition rating, however, is not the only measure of performance. In the US, the theoretical load carrying capacity of a bridge is evaluated and expressed in respect to a standard vehicle as load rating. Load rating may indeed be a better indicator of performance level as compared to condition rating. Firstly, it can be objectively calculated rather than by subjective evaluation. Secondly, it is in a continuous scale rather than a discrete scale so it can assume any real value. Thirdly, it is a ratio scale rather than an ordinal scale and thus is more amenable to mathematical manipulation. In fact, load rating is concerned with a different aspect of measuring bridge performance. It measures not only the capacity of structural members but indeed the capacity in relation to live loads. This way, it permits use of site specific information in its evaluation. In many instances, for example, in posting weight restriction on bridges load rating is a better criterion for decision making. It is logical that the relationship of the load rating with time should also be useful in managing existing bridge stocks. We call this type of relationship a load-rating based bridge deterioration model.

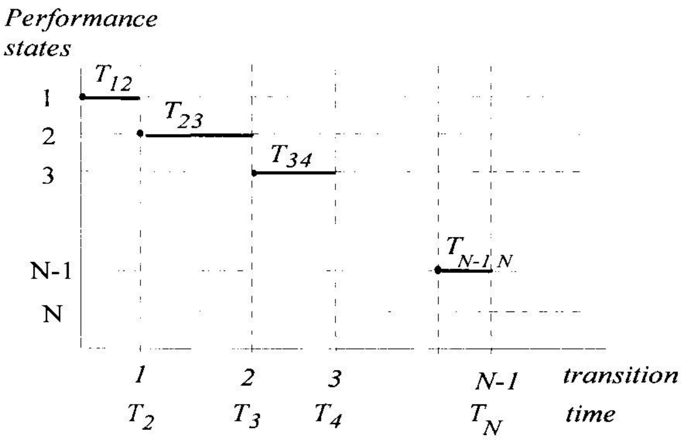


Fig. 1 Bridge Deterioration Process

2. The Modeling Procedure

A good discussion of bridge deterioration modeling can be found in [2,3]. Ref.[2] describes use of Markov model while ref.[3] considers a more general semi-Markov model. A semi-Markov process is a class of stochastic process that is governed by two different and independent random-generating mechanisms. When the process enters any state i, the probability of it moving to the next state j in the next transition is specified by the transition probability p_{ij}. Once the successor state is determined the process stays in the current state i for a duration T_{ij} which is dictated by the holding time probability density function $h_{ij}(m)$. In a bridge deterioration process the performance states would drop deterministically from one state to the next worse state; in a stepped function; as depicted in Fig. 1. Modeling of bridge deterioration process as a semi-Markov process therefore involves only the determination of the holding time distributions $h_{ij}(m)$.

Consider Fig. 1, if the probability distribution of the time to reach each state i (from state 1), viz., T_i is known then it may be possible to determine the distributions for holding times T_{ij} by taking the difference of T_j and T_i.

2.1 Time to state i, T_i

The time to reach a state T_i could be conceived as the time to failure or *failure time.* "failure" is used here to denote a distinct event, in this case, the reaching of the performance level represented by state i. The study of failure time has been the subject of a statistical analysis called survival analysis [4].

In traditional applications, *failure time* data is obtained from *life testing* (in industrial applications) or *clinical trials* (in medical applications). In either case, a specific number of the subjects of interest are observed for a period of time to obtain their individual times to failure (or time to the relapse of a certain decease). It is not uncommon to find that some of these items on test have been lost to follow up the study or have continued to survive at the end of the study period. As a result, the failure times of these subjects are not observed. The observations are said to be *censored.* Uncensored observations of failure times are called *complete* observations.

Bridges could not be subject to life testing for obvious reasons. A procedure proposed by Ng and Moses [3,5] use NBI (national bridge inventory) data for survival analysis. The data is treated as if obtained from a life test in which the addition of a new bridge in the inventory is regarded as an entry of the bridge in the life test. The ages of the bridge are regarded as an observed failure times. Whether an observed failure time is right-censored, complete or left-censored would depend on whether the reported performance is less than, equal or more than the specified state; respectively. By successively specifying the performance states $i = 2, 3, \ldots, N$ the time to reach each state (from state 1) can be derived using the procedure discussed above.

2.2 Holding time

Next is to determine the holding time distributions from knowledge of the distributions of T_i and T_j. Consider the difference of two arbitrary random variables $Z = X - Y$, the object now is to derive an expression for the random variable Z from the PDFs of X and Y. In particular, consider that Y is Weibull distributed with parameters (α_1, κ_1) and X is Weibull distributed with parameters (α_2, κ_2). Ng & Moses [3] assumes that the number of transitions taking place in the deterioration process is governed by a nonhomogeneous poisson process called quasi-Weibull process. This process is a naïve adaptation of a Weibull process. Some discussion of Weibull process is given in [6]. Based on this assumption the PDFs of Z can be obtained from Eq. 1.

$$f_Z(z) = \int_z^{\infty} \frac{\alpha_1\alpha_2\kappa_1\kappa_2(x-z)^{\kappa_1-1}x^{\kappa_2-1}\exp\left[-\alpha_1(x-z)^{\kappa_1}\right]}{\exp\left[-\alpha_2(x-z)^{\kappa_2}\right]}\cdot\exp[-\alpha_2 x^{\kappa_2}]dx$$

$$= \alpha_1\alpha_2\kappa_1\kappa_2\int_z^{\infty}(x-z)^{\kappa_1-1}x^{\kappa_2-1}\cdot\exp\left[-\alpha_1(x-z)^{\kappa_1}-\alpha_2[x^{\kappa_2}-(x-z)^{\kappa_2}]\right]dx \quad (1)$$

By letting $Y = T_i$ and $X = T_{i+1}$, the PDFs of the holding time, $h_{i+1}(m)$ for $i = 1, 2, \ldots N$ can be computed using Eq. (1).

3. Connecticut Model

The NBI data (1989-1993) from Connecticut, U. S. A. was used to illustrate the procedures described above. The bridges were divided into their respective groups in terms of their construction materials and design load. A separate model was developed for each group of bridges. Steel bridges designed to HS20 (Group C) constitute majority of the bridge stock, i.e., 81.13% and will be reported here.

Load rating represents the load-carrying capacity of a bridge in terms of the gross weight (in tons) of standard vehicles measured in a continuous scale. To use the semi-Markov representation the scale is arbitrarily discretized into 11 states as in Table 1. For more exact modeling of the continuous scale more states could of course be used.

Table 1
Performance States Defined

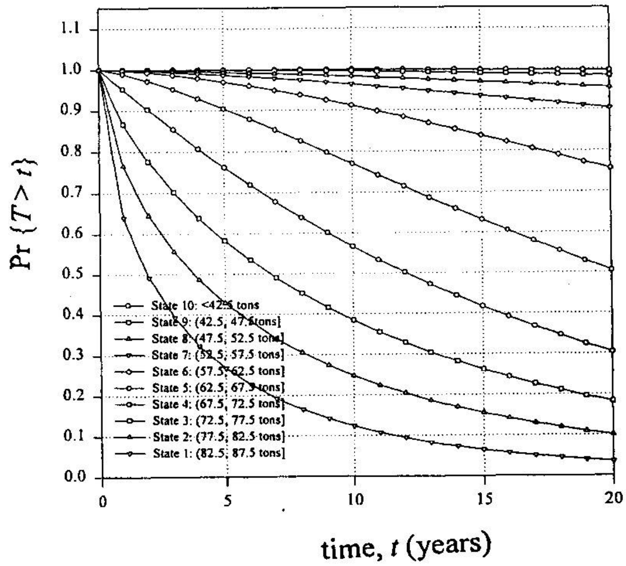
Performance state, ξ	Load rating at opertaing level (tons)	Performance state, ξ	Load rating at opertaing level (tons)	Performance state, ξ	Load rating at opertaing level (tons)
1	> 97.5*	5	(67.5, 72.5]	9	(47.5, 52.5]
2	(82.5, 97.5]	6	(62.5, 67.5]	10	(42.5, 47.5]
3	(77.5, 82.5]	7	(57.5, 62.5]	11	< 42.5
4	(72.5, 77.5]	8	(52.5, 57.5]		

4. Results

Survival analysis essentially fit the failure time data to a probability distribution; very often, in the form of survival function. A survival function is defined as $\Pr\{T > t)$, that is, the time function of the probability that the time to failure T exceeds time t. In this example, the data was fit with Weibull distribution and the goodness of fit was found to be good. SAS [7] procedure *lifereg* was used to estimate the Weibull parameters and the results obtained were presented in Table 2.

Tables 2
Results of Survival Analysis

Load	*Group C: steel, design load = HS20*	
Rating (tons)	Scale parameter α	Shape parameter κ
(82.5, 87.5]	0.4464713	0.67125627
(77.5, 82.5]	0.26666335	0.71838668
(72.5, 77.5]	0.14258163	0.82704777
(67.5, 72.5]	0.04877082	1.06634705
(62.5, 67.5]	0.01074504	1.38615893
(57.5, 62.5]	0.00235123	1.59552996
(52.5, 57.5]	0.00133146	1.4472555
(47.5, 52.5]	0.00061842	1.44524961
(42.5, 47.5]	5.46E-05	1.92116315
<42.5	4.62E-07	3.06357531



The parameters in Table 2 were used for plotting the survival functions as presented in Fig. 2.

Fig. 2 Survival Functions

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* Not including 99 because the entry of '99' in the NBIcoding represents "rating not available."

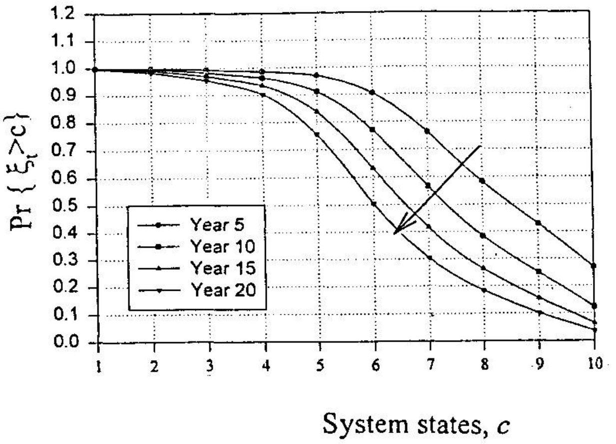

Fig. 3 Complementary cdf of Load Rating

The family of survival functions in Fig. 2 together contain much information about the deterioration process of Group C bridges. Each survival function represents the probability distribution of the time to reach a certain performance state. Eq. (1) is then used to determine the holding time distributions as discussed in Section 2.

It would be interesting to examine the survival functions from another angle. If we were to cut across the family of survival functions in Fig. 2 at any time t we would obtain a function which is the cumulative distribution function (cdf) for random performance state. The performance state at time t = 5, 10, 15 and 20 years are separately treated here as a random variable whose complementary cdf's are plotted in Fig. 3. The graphs exhibits a shift of the performance state distribution due to the passage of time from year 5 to year 20.

5. Applications

One major application of a bridge deterioration model is the prediction of future performance. For performance prediction (under no human intervention), Eq. (2) is applicable [2].

$$\pi(m) = \pi(0) \times \{\ \psi_{ij}(m)\ \} \qquad (2)$$

where $\pi(m)$ is the state probability vector at time m and$\{\ \psi_{ij}(m)\ \}$is the *interval probability matrix*. The element $\psi_{ij}(m)$ is the probability that the process will occupy state j at time n given that it entered state i at time zero. Consider m as the time for the first transition to occur after the process entered state i, ref. [2] gives two equations for the determination of $\psi_{ij}(m)$. For $m < n$, the interval probability is given in Eq. (3). For $m > n$, Eq. 4 applies.

$$\psi_{ij}(m) = \sum^{n}_{m=1} h_{ii+1}(m)\ \psi_{ij}(n-m),\ j \neq i \qquad (3)$$

$$\psi_{ij}(m) = 1 - {}^{\leq}h_{i\,i+1}(m),\quad j \neq i \qquad (4)$$

where the notation ${}^{\leq}h_{i\,i+1}(m)$ is used to denote the cumulative distribution function of the holding time $h_{i\,i+1}(m)$.

Using Eq. (3) and Eq. (4) the interval probability for each i-j pair was computed using a computer software called *Mathematica* [8]. Only the matrix for m = 10 years are presented in Table 3.

Table 3 Interval Probability Matrix, $\Psi(m)$

$\Psi(m) = \{ \psi_{ij}(m) \}, i = 1, 2, ..., 11; j = 1, 2, ..., 11$

m=10

0.203	0.418	0.279	0.086	0.012	0.001	0.000	0.000	0.000	0.000	0.000
0.000	0.399	0.394	0.171	0.033	0.003	0.000	0.000	0.000	0.000	0.000
0.000	0.000	0.478	0.386	0.120	0.015	0.001	0.000	0.000	0.000	0.000
0.000	0.000	0.000	0.550	0.366	0.078	0.005	0.000	0.000	0.000	0.000
0.000	0.000	0.000	0.000	0.637	0.323	0.039	0.001	0.000	0.000	0.000
0.000	0.000	0.000	0.000	0.000	0.769	0.221	0.010	0.000	0.000	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.904	0.094	0.002	0.000	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.945	0.054	0.001	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.046	0.004
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.817	0.183
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.000

6. Conclusions & Recommendations

This paper implements two new ideas in bridge deterioration modeling. They are:

i. Use of load rating in deterioration modeling;
ii. Use of semi-Markov formulation;

The use of load rating in the deterioration models allows bridge management decisions which are based on load rating. Examples are bridge posting, issuance of special permits for oversized vehicles. Also, the load rating-based deterioration model facilitates use of structural reliability theory in project-level bridge management decisions.

One possible extension of this model is the use of continuous scale for performance level instead of arbitrarily divide the scale in discrete states.

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Design and Detailing for Durability – Concrete Subways and Underpasses

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Summary

Durability of structures refer to its ability to endure weathering action, attack by various substances and conditions to which it may be exposed over the serviceable life. Many of the conditions that cause structures to lack durability, are not immediately apparent. Basic causes are attributable to input of harmful materials, aggressive environment or inappropriate design, detailings or construction. This paper discusses design and detailing features for pedestrian and vehicular underpasses, under construction, to improve durability aspect. As such, approach to design and detailing for three cases of subway/underpass projects in India, was to design for durability, faster implementation and simplied construction technique. Projects discussed in this paper are pedestrian subway at Rajagarden, integrated pedestrian & vehicular underpass at Punjabi Bagh in Delhi and underground mass rapid transit system, in Calcutta.

1. INTRODUCTION

Electrically operated intracity Mass Rapid Transit System, whether underground, at grade or elevated is rare in India. Such facilities, to a limited extent, have been built very recently in few cities and some are still in planning/implementation stages. At present transport system in India, primarily cater for motorised vehicles both for mass and individual transport. Most of the major intersections and corridors of the major cities in India are very congested. To ease such congested situations, transport authorities are planning for multilevel grade separators for pedestrian and vehicular movements and also to increase the average speed of movement. Such grade separated movement warrants pedestrian subway, or vehicular underpass. Their planning and design however need to cater for existing underground utilities, narrow roads, lack of space for traffic diversion ground water table etc.

In spite of appropriate and detailed implementation planning and schedules, with cast-in-situ construction, pedestrian subways need at least twelve months for completion, which is too long a period to have restricted traffic movement in an intersection. In order to minimise the traffic hindrance, reduce construction period & on site activities, to improve the quality of construction, etc, the use of prefabricated concrete elements has been introduced for Rajagarden pedestrian subway at Delhi as shown in **Fig. 1**.

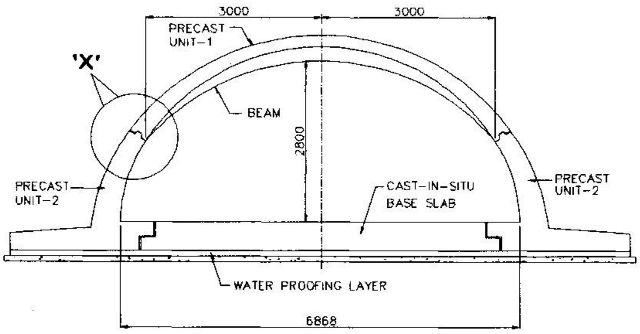


Fig. 1 : Precast Subway showing different Units

For the construction of Punjabi Bagh underpass or for that matter the construction of Calcutta Metro, use of modern systems e.g. TBM or NATM might have been a better choice particularly to maintain normal traffic movement during construction. However, as construction using TBM or NATM is very expensive and need capital intensive equipments, cut and cover technique are more favoured bothfor vehicular underpass or metros.Out of the different forms of temporary/permanent walling, diaphragm wall has been used predominantly.

For underpasses in Delhi or Metro in Calcutta being constructed or constructed, diaphragm walling have been used not only to facilitate other construction activities but also to bear load either in partial or/in total as the case may be i.e. semi-integrated or integrated as shown in **Fig. 2** and **Fig. 3.**

Parameters for design, detailing and construction works method have been carefully selected in these designs and are briefed in the following sections.

2. CONSTRAINTS AND APPROACH

During the planning for design and construction of these structures, number of constraints were taken care of proper implementation as :

- High ground water table
- Existing major underground utilities
- Limited space available for construction in busy traffic corridor

- To keep the intersection traffic moving during construction phase
- Minimise construction time to open the carriageways as soon as possible for the convenience of traffic.

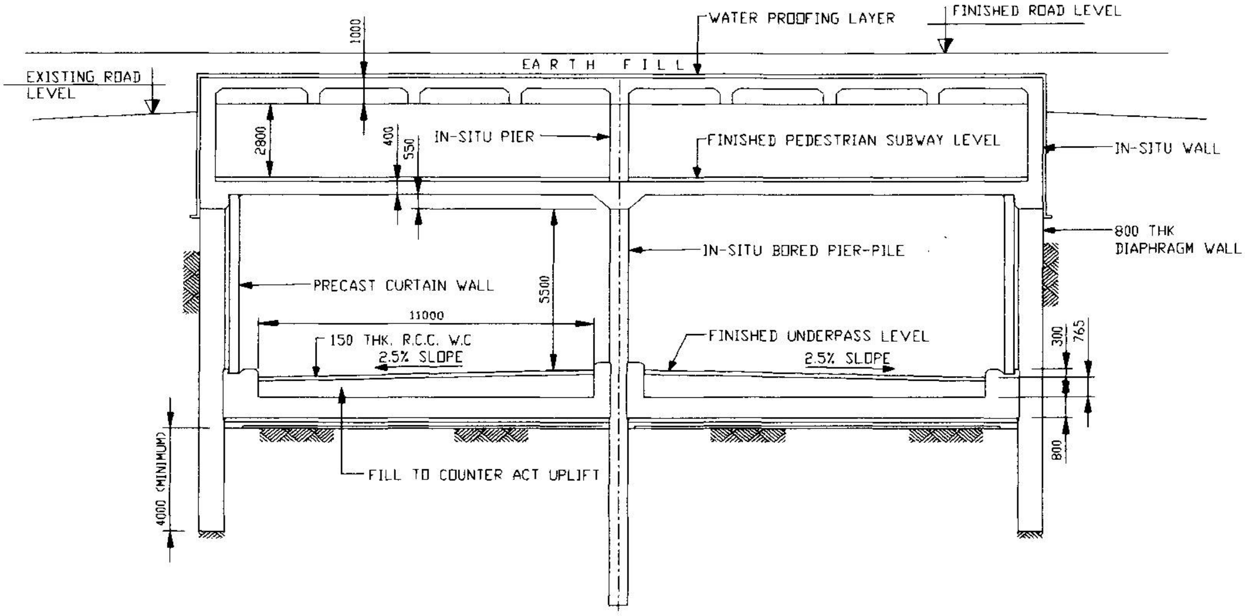


Fig. 2 : Cross Section of Underpass and Pedestrian Subway at Intersection

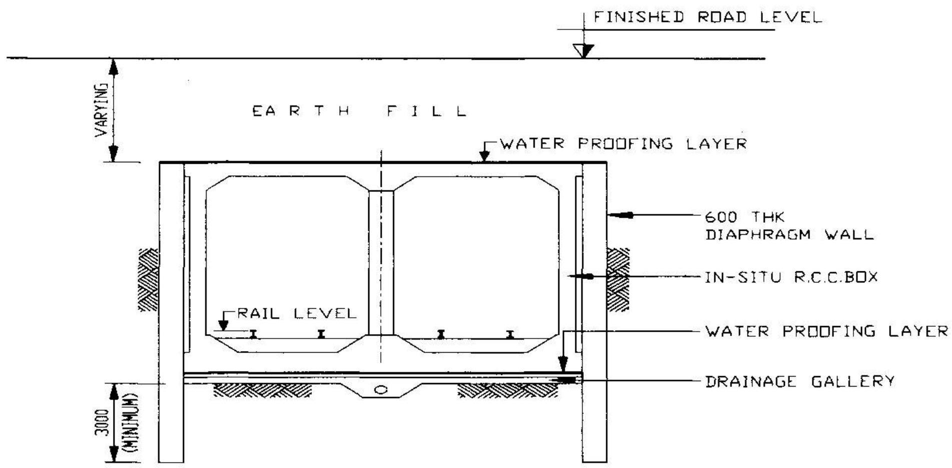


Fig. 3 : Cross Section of Metro : Non-Station Portion (Semi-Integrated Approach)

In addition, as these underground structures, with a high ground water table warrants dry condition inside the underpass or subway, for serviceability and user's comfort, the following aspects were also taken care in the design:

- Water tight joints & adequate water proofing treatment
- System of draining out the seepage and/or rain water from open areas
- Adequate ventilation, forced or natural, particularly for pedestrian subway

Considering the constraints stated above and also the small length of such subways or underpasses, the design and construction methodology adopted, caters for faster implementation without capital cost intensive equipments, minimum possible hindrance to traffic movement during construction and water tight structures to the extent possible including appropriate drainage system.

3. PEDESTRIAN SUBWAY

As discussed earlier, for faster implementation with minimal capital investment on equipment to make the project cost effective and to ensure quality and durability, prefabricated reinforced concrete element has been used for the pedestrian subway and the design considered following important aspects :

- Excavation in stretches to allow traffic movment along the road
- Form to suit for structural stability, drainage, light weight and cost effectiveness
- Element Size to suit transportation and handling
- In-situ concrete base to ensure water tightness
- Proper integration between precast concrete elements and cast in situ elements
- Matching of individual precast element and design of joints both for installation tolerances as well as water tightness
- Appropriate drainage arrangement
- Provision against buoyancy

The pedestrian subway is approximately 40 m long with central opening. Subway form designed as catenary mainly comprising of precast element (**Fig. 1**). The top precast unit (unit 1) was provided with a central diaphragm of varying depth for handling facilities. However for future subways this beam is proposed to be excluded as these ribs create hindrance for installation of lights and electrical cable ducting.

As elaborated in **Fig. 4** the joints have been detailed for easy matching installations with provision for water tightness. **Fig. 5** gives the details of match installation of the precast units in longitudinal directions with provision for water tightness.

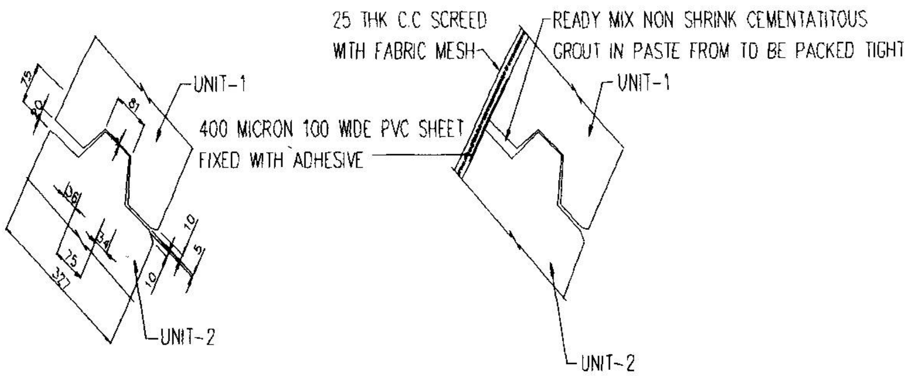


Fig. 4 : Detail 'X' (Ref. Fig.1)

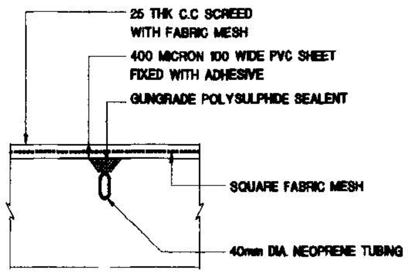


Fig. 5 : Typical Joint between Precast units in Longitudinal Direction

Fig. 6 indicates the methodology of water tight integration of the longitudinal subway at its ends with the approach stairs in the perpendicular direction.

For the integration of the bottom slab with the precast unit (**Fig. 1** and **Fig. 7**), the same has been shaped and detailed such that the connection can be treated as hinge with provision for rotation of the ends of the bottom slab at its ends. The reinforcement detailing has been done to ensure the hinge behaviour.

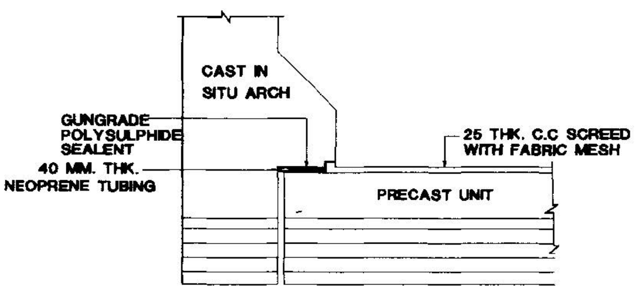


Fig. 6 : Details of Joint between Precast Unit and Cast-in-Situ Arch

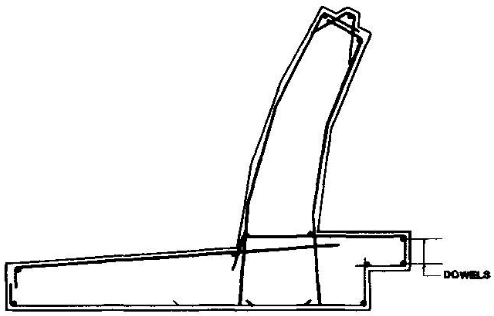


Fig. 7 : R C Details of Unit - 2

Number of prefabricated elements being small, precasting of the elements are done near the site to avoid long/distant travel during transportation. As such elements can be easily transported by a mobile crane and each element can be easily installed in proper place with integration of the other elements. Use of various methods as detailed in sketches ensure water tight subway. However provision for a sump with pumping arrangement has been kept for draining out of the rain water that will pour inside the subway through the open approaches.

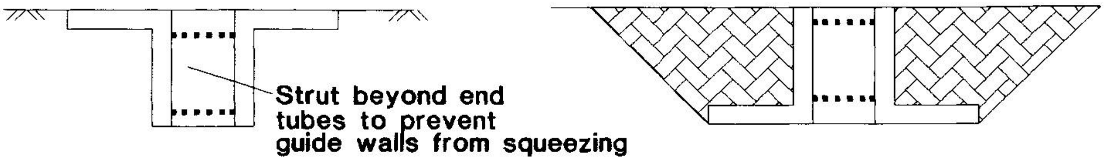
4. UNDERPASS

As mentioned earlier, the cases considered are Calcutta Metro and integrated pedestrian and vehicular underpasses in Punjabi Bagh, New Delhi. As the design approach for both the cases are more or less similar except the changes required for functional performance as semi integrated and fully integrated structural system. The important aspects which are directly related to the serviceability and durability are described in the subsequent sections.

4.1 Guide Walls

The guide walls are not an integral part of the underpass system, yet they are important in excercising control on the construction of the diaphragm walls. The guide walls should be heavy with appropriate depth and contractor should take proper care of the compaction of filling after completion of the guide walls. Accordingly, the design of the guide walls in both the cases are to consider the following aspects :

- Appropriate depths to avoid excessive and cavity formation
- Due consideration to ground water table
- Appropriate head of bentonite slurry with proper viscosity and gel strength
- Avoid movement of the guide walls to ensure verticality of diaphragm wall
- Proper orientation of the guide wall as shown below



Alternative A

Alternative B
(Preferred alternative)

Fig. 8 : Guide Walls

Accordingly, the depth of guide walls in the range of 1.2 to 1.5 m have been adopted.

4.2 Panel Width

Concrete diaphragm walls are generally constructed in panels, wherein each panel acts as an independent unit. While selecting the size of the panel width in both cases the Calcutta Metro with clayey strata and Punjabi Bagh underpass with granular strata, attempt was made to mobilize the strut forces as per the design load.

The panel width has been kept to a size so that one row of horizontal strut only for each panel will be sufficient for the safety of excavation. However the panel width can not be less than the single bite widths of grab. Panel width of approx. 3 m has been found to be cost effective and technically optimum.

In addition, while selecting the size of the panels, the following points had been considered :

- Continuous diaphragm walls need Robust shear connectors which are difficult to provide.
- Concreting have to be completed before the setting or significant stiffening of the panel (Not more than 3 to 3 ½ hours)
- Removal of stop-ends before the concrete is set.
- To reduce the formation of shrinkage crack (Though the curing is under humid conditions the longer panel lengths will be subjected to shrinkage effects)
- Safety of the excavated trench and easy handling of the reinforcement cage.

4.3 Water Tightness

Diaphragm wall are subjected to both soil and water pressures and thereby impermeability of wall panels is of great concern for durable structural system. The various factors affecting the water tightness are :

- Quality of the concrete with regard to maximum aggregate size and water cement ratio (20 m aggregate & w/c ratio 0.6)
- Formation of cracks particularly for longer panels

- Leakage from the joints attributable to differential deflection between the panels
- Semi circular vertical joints at ends of each panel

Sometime shear transfer devices are used to help in reducing the differential deflection between consecutive panels and thereby improves water tightness. But to provide shear connector between two panels is difficult and combersome. Accordingly, in most of the underpasses or metro, water has been allowed to seep through the joints.

The leakage through joint was not critical for Calcutta Metro as the water tightness was ensured through second wall. While in Delhi, cement grout has been proposed at the back of the joint to ensure water tightness to the extent possible as shown in **Fig. 9**. In addition an elaborate arrangement for drainage has been made to keep the underpass dry, serviceable and durable as shown in **Fig. 10a** and **10b**.

4.4 Buoyancy

As the ground water table is very high in both the cases the structure had to be designed to resist the uplift due to buoyancy forces. In Calcutta Metro this was not critical in most of the stretches excepting the station areas, primarily due to overburden on top of the box as shown in the figure.

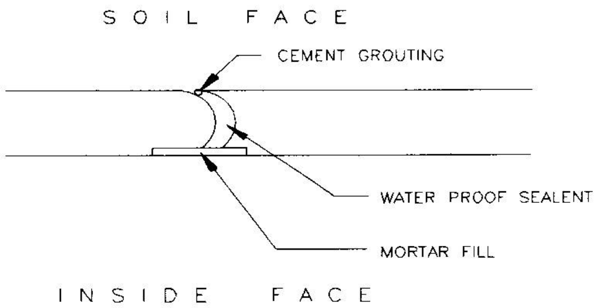


Fig. 9

In case of Punjabi Bagh underpass the uplift becomes highly critical particularly in open to sky approaches due to the absence of adequate over burden. Alternative approaches has been considered to ensure stability

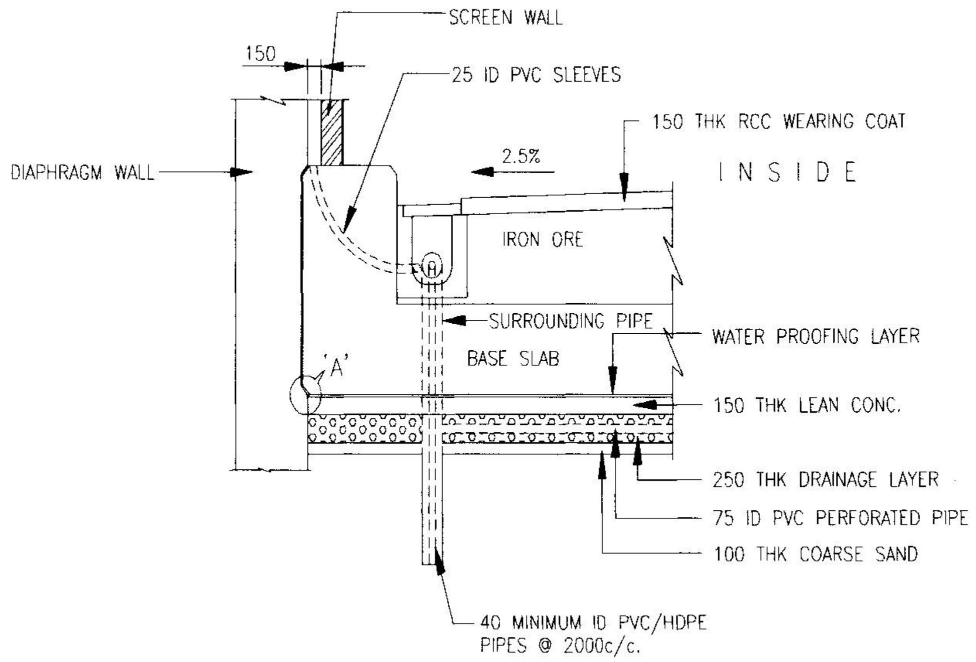
- To put additional weight in the form of iron ore filling
- To provide tension piles to resist the uplift forces.
- To release the water pressure by providing suitable collection of underground water and draining thereof.

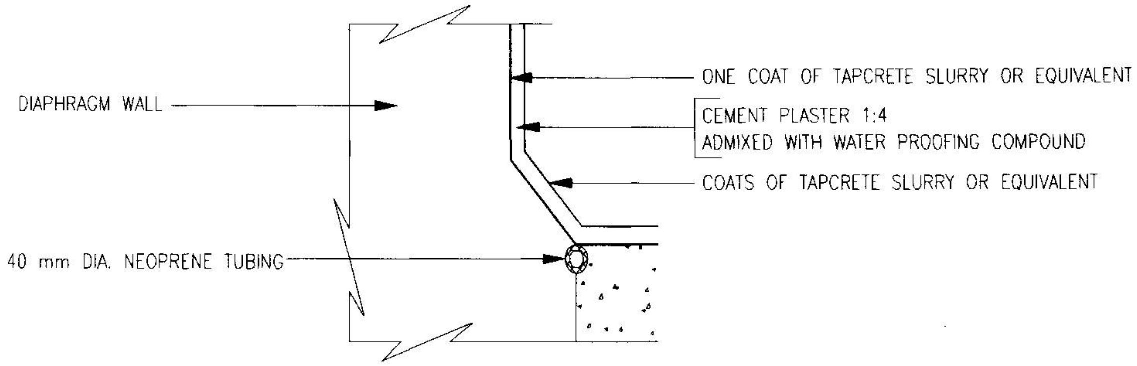
Each systems has its own merits and demerits though the authors favour the combination of the last two

4.5 Reinforcement Details

The design and detailing of the reinforcement caging shall take into consideration the following aspects :

- Smooth bar bending with adequate cover
- Bar placement to avoid the trapping of bentonite slurry
- Proper securing of the insert boxes avoiding any protusion
- Appropriate detailing to cater for the handling





DETAIL 'A'
DETAILS OF DRAINAGE SYSTEM FOR UNDERPASS

Fig. 10a and 10b

5. CONCLUSION

Experience shows that improvements in standard of controls has a direct effect on improving the standard of the finished product and thereby its durability. Thus the various aspects concerning the durability, serviceability of the specific project shall be detailed out at the project conception stage prior to design and detailing.

A Performance-Based Evaluation for a Reinforced Concrete Member Subjected to Chloride Ingress

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Summary

A framework of a performance-based evaluation of a reinforced concrete member subjected to chloride ingress is summarized. Chloride profile in cover and the propagation of chloride-induced corrosion of a rebar embedded in the concrete are quantified. The amount of corrosion products which would increase with time is used to calculate the time for a longitudinal crack due to the corrosion to occur and its width. The width of the longitudinal crack is then used as an index for objective judgement in evaluating the performance of the concrete member.

1. Degradation Process

A steel reinforcement in concrete will rust after chloride ions penetrating in the cover reach its surface and accumulate at a threshold concentration. The cross sectional area of the corroding rebar is reduced with the progress of corrosion resulting in the deceased flexural and/or shear strength of the member. Therefore, the corrosion of a steel reinforcement can affect adversely the safety and serviceability of concrete structures and hence shorten the service life.

The performance of a concrete structure should be examined in a time-dependent phenomena where respect mechanisms involved may need to be quantified on a rational basis(1). Fig.1 shows a progressive loss of the performance of a concrete member subjected to chloride ingress. The degree of the corrosion of a steel-reinforcement is responsible for its performance. Each periods in Fig.1 are normally explained as follows:

(I)Initiation period(Δt_1) Penetration of chloride ions in concrete and the accumulation on the surface of a rebar to a threshold concentration

(II)Propagation period(Δt_2) Progression of corrosion with oxygen and water supply

(III)Acceleration period(Δt_3) Advance of corrosion due to increased corrosion rates after the formation of longitudinal cracks and loss of structural performance due to decreased cross section of rebar

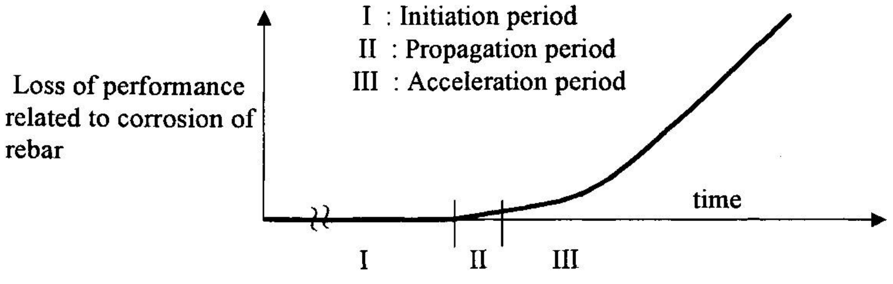

Fig.1 Time-dependent change in the performance of concrete structures with chloride ingress

2. Methodology of the Quantification of Chloride-Induced Corrosion

Several methodologies have been proposed to predict each period in a given stage. Particularly, a number of mathematical models have been used to predict an initiation period. Mathematical models to calculate the period of an initiation stage on the basis of the Fick's first and second laws were introduced in a state-of-the-art report published from the JCI research committee in 1996(2). It may be appropriate at present to assume that the movement of chloride ions through concrete follows the diffusion theory.

Propagation and acceleration periods may be calculated with the corrosion rate of an corroding reinforcement in cover. The corrosion rate is normally calculated on the basis of the electro-chemical theory. A state-of-the-art report on related subjects about the corrosion of rebar has been published from a research committee in the JCI in 1996(3) and in the JSCE in 1997(4).

It must be acknowledged that the rate of chloride penetration through concrete and the corrosion rate of corroding reinforcements are calculated with a large number of assumptions and simplified models for the time being. Future works are definitely needed to verify assumptions and hence modify analytical models used to increase the precision of the prediction of the performance of a concrete structure subjected to chloride ingress.

2.1 Initiation periods, Δt_I

A diffusion model may be used to calculate an initiation period. A governing equation for chloride ions diffusion in concrete may be given by the Fick's first law. On the other hands, for calculating the profile of chloride content in concrete the Fick's second law is normally used for the mass conservation. In addition to diffusion process, the transport of chloride ions may occur in flowing water. Especially, when wetting condition arrives at concrete surface after drying cycle, a large amount of moisture is absorbed into pores due to the capillary suction. Therefore, chloride transport through concrete may follow a global equation given below(5):

$$\frac{\partial C_t}{\partial t} = -div\left[D_a \cdot \nabla(C_f)\right] + C_f \underline{J}_w \qquad (1)$$

C_t : total amount of chloride content in concrete (kg/m^3), C_f :the amount of free chloride ions in pores (kg/m^3), D_a : nominal diffusion coefficient (cm^2/s), $\underline{J}_w$: moisture flux(1/s)

In addition to quality of concrete which is controlled by mix proportions, materials and constructions, environmental conditions have significant effects on the transport of chloride ions through concrete. The total content of chloride ions is a product of a free chloride concentration in pore solution added to the amount of chlorides bound in cement hydrates. Then, the binding isotherm of chloride ions in concrete is important, which is primarily controlled by binder types, its amount and concrete age. The diffusion coefficient of chloride ions may be controlled by a microstructure, moisture profile and temperature in concrete. Furthermore, the boundary condition for the differential equation shown in Eq.(1) must be properly determined. For a diffusion equation the change of surface concentration of chloride ions (C_s, $x=0$) with time, t is normally used as the boundary condition, i.e. $C= C_s$, $x=0$, $t \geqq 0$ for one dimensional analysis. Therefore, it is largely controlled by such environmental conditions as the amount of airborne salt, rainfall, relative humidity, the amount of deicing agent if used during winter season, temperature and so on. For concrete submerged in sea water, the surface concentration of chloride ions may be constant while it is fluctuated with time for concrete above ground due to rain, carbonation and the periodic change of the amount of airborne salt. Thus the determination of the boundary conditions is complex. The survey of concrete structures above ground in chloride-laden atmospheres shows that the concentration of chloride ions near the surface area, about 0.5 cm deep from the surface, is changed with time(4) and may be modeled as follows:

$$C_s = C_0\{1 - \exp(-\beta t)\} \qquad (2)$$

C_0 and β are constants and should be determined by an unique characteristic in service environments and quality of surface concrete. Therefore, Eq.(2) may be used as the boundary conditions for concrete above ground.

For a reinforced concrete subjected to flexure, flexural cracks are allowed to occur within a limited width. However, the transport of chloride ions through a crack in concrete is difficult to be quantified. For the time being, the average transport of chloride ions in a cracked concrete is modeled with an increased diffusion coefficient of chloride ions or an increased void ratio.

2.2 Calculation of Propagation Period, Δt_2 and Acceleration Period, Δt_3

Propagation and acceleration periods may be calculated with the amount of corrosion product and corrosion rate. This is because the development of a longitudinal crack is a time-dependent process and related closely to an increased amount of corrosion product with time.

When an expansive stress induced by corrosion reaches the tensile force of cover concrete, longitudinal cracks will occur. Then, a propagation period may be calculated as follows:

$$\Delta t_2 = \frac{W_{cr.}}{VW_{corr.}} \qquad (3)$$

Where W_{cr} is the amount of corrosion product at the occurrence of longitudinal cracks in mg/cm^2 and VW_{corr} is the corrosion rate in mg/cm^2 /year.

Corrosion current is necessary to calculate the corrosion rate and may be obtained from a current circuit model. Corrosion current density is generally expressed as follows:

$$I_{corr.} = \frac{\Delta E}{r_A + r_C + \frac{r_{Con.}}{k}} \qquad (4)$$

Where I_{corr} : corrosion current density in mA/cm^2, ΔE: : potential difference between anode and cathode in mV, r_A and r_C : anode and cathode polarization resistance, respectively in $\Omega \cdot cm^2$, r_{Con} : concrete resistivity in $\Omega \cdot cm$, k : cell constant in cm^{-1}.

Normally chloride-induced corrosion is in the process of macrocell corrosion where the anode area is created by chloride accumulation while the cathode area is created far from the anode and is supported by the oxygen. The Faraday's law can be used to calculate corrosion current with the amount of oxygen reduced at the cathode, where the corrosion area must be specified. The resistivity of concrete is also necessary for corrosion current, which is converted to a corrosion rate. The amount of corrosion product increases with the continuous flow of corrosion current. In concrete structures which suffer from the chloride attack in a marine atmosphere, the relative humidity in the cover is enough high to motive the corrosion process that is controlled by the diffusion rate of oxygen. Therefore, the diffusion of oxygen in concrete and its amount available for the reduction at the cathode is important to determine the corrosion rate(1). Corrosion rate increases to the maximum when the relative humidity in cover concrete ranges from 60 to 80 %(4). In concrete structures submerged in sea water, little oxygen can transmit to the surface of rebar and hence the corrosion rate is low. In addition, corrosion rate may be negligibly low for dry concrete although oxygen can diffuse easily. The resistivity of the concrete becomes higher resulting in the reduction of the corrosion current. Therefore, the magnitude of concrete resistivity controls the corrosion process in the dry concrete. The ratio of anode to cathode area are thought to decrease with time and hence the corrosion rate is a time-dependent process(1).

The amount of corrosion product, W_{cr} at the occurrence of longitudinal cracks can be calculated with the applied linear elastic analysis for a thick-wall cylinder(1). Therefore, physical characteristics of a corrosion product and the arrangement of reinforcements in concrete play an significant role as well as the tensile strength of concrete. In addition, drying and shrinkage and dead and live loads applied are responsible partially to the formation of longitudinal cracks. Few studies of these effects on crack formation have been carried out. Furthermore, cover concrete may be spalled off with a given amount of corrosion product. However, the spalling of cover concrete may be related to the arrangement of reinforcements and their diameter [4].

Although a corrosion rate after a longitudinal crack occurs is difficult to be quantified because of insufficient knowledge of its mechanism and the lack of field data, acceleration period will be calculated as follows:

$$\Delta t_3 = \frac{W_{corr.}}{VW_{cr.}} \qquad (5)$$

Where $W_{corr.}$ is the amount of corrosion product necessary for a given crack width in mg/cm^2 and VW_{cr} is the corrosion rate after the occurrence of longitudinal cracks in mg/cm^2 /year.

Laboratory studies show that with a given diameter of rebar and cover depth, a crack width appears to increase linearly with the amount of corrosion product. However, there are few studies on the corrosion rate after the occurrence of longitudinal cracks. Therefore, the result of the corrosion rate obtained from experiments where bare bars have been exposed directly to marine environments may be used.

3 Sample calculation

A performance-based evaluation of a reinforced concrete beam exposed to a chloride-laden atmosphere is conducted. The required performance is assumed in this particular example that the width of a longitudinal crack due to corrosion of rebar is less than 0.8mm for considering its serviceability. Construction works are assumed to be carried out properly according to a construction manual so that unfavorable effects on the chloride penetration can never exist.

3.1 Quantified Environments

Periodic change in temperature and air borne salt are shown in Fig.2 and Fig.3. The amount of airborne salt increases in winter season since relatively high ocean wave occurs due to the wind heavily blowing. Other meteorological data is provided in Table 1.

Table 1 Meteorological data

Temp.	RH in average.	Airborne salt	Rainfall
Max.25 ℃ Min. 2 ℃	65 % 80% (microclimate effects)	Max. 0.62 mg-Cl/cm^2/day Min. 0.62x10^{-2}mg-Cl/cm^2/day	1200mm/ year

If a river flows underneath the bridge wet conditions may be maintained in the bottom part of the beam. Also, its face is hardly subjected directly to the sunshine and heavy rain- shower. These microclimates are important to the transport of chloride ions and are taken into account to calculate RH in the concrete cover and is assumed to change with time (see Fig.4).

3.2 Calculation of Chloride Penetration

Initiation period is calculated with Eq.(1) where $C_f J_w$ is neglected since the transport of chloride ions dragging into flowing water due to absorption is limited near surface area in this particular example. For the boundary condition Eq.(2) is used where constants, Cs and β, are properly determined from environments, especially the amount of airborne salt and the annual rainfall. Then the boundary condition is assumed in Fig.5. Chloride diffusion coefficient changes with increased hydration periods the periodic change of temperature and relative humidity in cover. Also, the porosity of concrete surface may be different from the porosity of the inner concrete due partially to a curing method. The binding isotherm of chloride ions can be referred from the past study of similar materials[6]. The flexural cracks will be present on the tension region of the beam. Its effect is taken into account by increasing the average porosity of the cover.

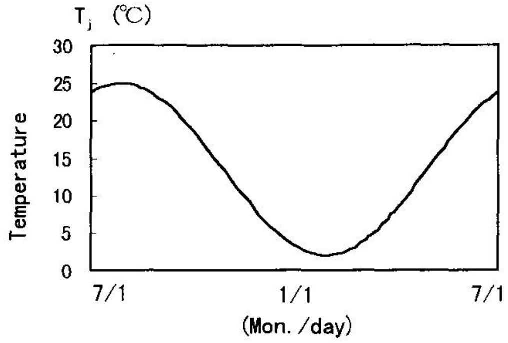

Fig. 2 Seasonal change of temperature

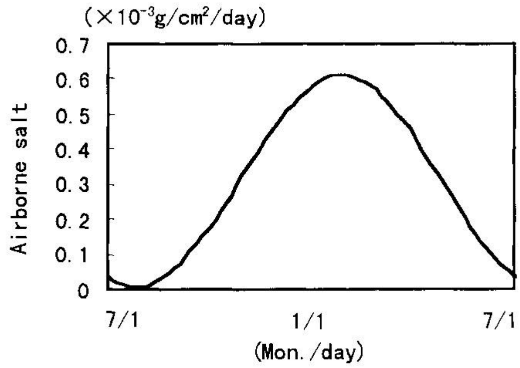

Fig. 3 Seasonal change of airborne salt

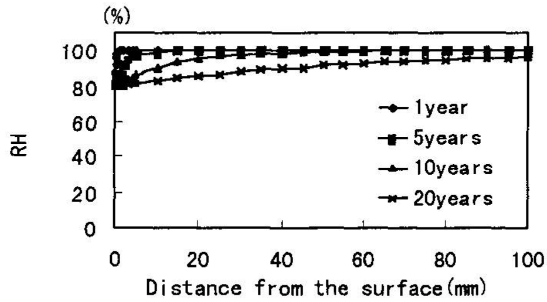

Fig. 4 Decrease of relative humidity in cover

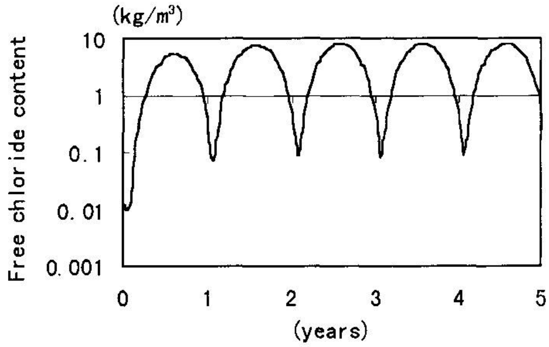

Fig. 5 Boundary conditions (x=0,t>0)

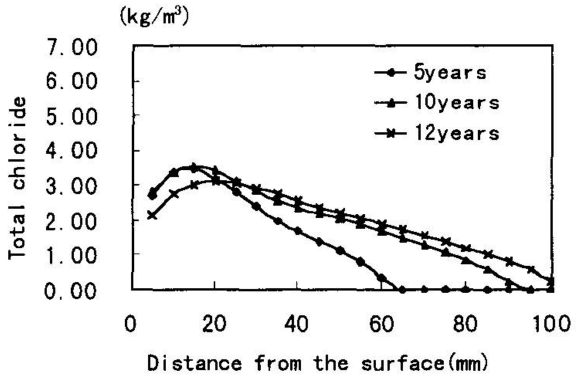

Fig. 6 Chloride profile in cover

The profiles of chloride concentration in concrete cover after 5, 10 and 12 years are shown in Fig.6. A threshold content at rebar surface for chloride ions to initiate corrosion is 1.2 kg/m^3 on the basis of total amount of chloride content. Also, the cover depth is equal to 84 mm. Therefor the chloride content at an depth of 84 mm reaches 1.2 kg/m^3 at an period of 12.4 years. It means that the steel embedded in the cover starts to rust at an service period of 12.4 years.

3.3 Calculation of Corrosion Rate

To calculate the corrosion rate Eq(3) is used. However, the prediction of the corrosion rate would be more difficult as compared with the chloride analysis at this present. Then, each values necessary for the calculation are determined from experimental data[(4)] and summarized in Table 2. The corrosion rate results in 28.2 mg/cm^2/year. In the calculation the concrete quality and environmental conditions unique to this particular case should be taken into consideration.

A linear elastic analysis[(3)] is conducted to calculate the amount of corrosion at the occurrence of longitudinal cracks. Cover concrete responses elastically against expansion force induced by

Table 2 Values for the calculation of the corrosion rate and the results

ΔE:	r_A	r_C	r_{Con}
300 mV	0 (due to effects of flexural cracks)	80 kΩ·cm²	13 kΩ·cm
I_{corr}	i_{corr} (corrosion current)	VW_{corr}	W_{cr}
3.1 μA/cm²	49.7 μA	28.2 mg/cm²/year	15.8 mg/cm²

corrosion product. A crack occurs when the tensile stress induced by the expansion reaches the tensile strength of cover concrete. This corrosion amount results in 15.8 mg/cm². Therefore, the propagation period is 0.56 years.

3.4 Calculation of Crack Width with the Amount of Corrosion

The corrosion rate after the occurrence of longitudinal cracks is determined on the basis of experimental data(1) where bare bars have been exposed to a tropical marine atmosphere for 3.5 years. Then, it is assumed that the corrosion rate would increase with square root time as shown in Table 3. In addition, the width of the longitudinal crack would increase linearly with the amount of corrosion product and the equation for this particular example is shown in Table 3 where constants are determined with bar diameter and cover depth.

Table 3 Corrosion rate after longitudinal crack and relationship with the crack width

VW^*_{cr}	W_{corr}	$W_{corr} = a\, w_{cr} + b$
$86\sqrt{\Delta t_3}$ mg/cm²	74 mg/cm²	$W_{corr} = 80\, w_{cr} + 10$

VW^*_{cr}: the corrosion rate after longitudinal crack is modeled in a square root time relation.

3.5 Summary of the Sample Calculation

The results of the initiation, propagation and acceleration periods are summarized in Table 4.

Table 4 Results of the initiation, propagation and acceleration periods

Δt_1	Δt_2	Δt_3	$\Delta t_1 + \Delta t_2 + \Delta t_3$
12.4 years	0.56 years	0.74 years	13.7 years

This shows that after a service period of 13.7 years the width of a corrosion-related longitudinal crack reaches 0.8 mm. It seems that the results of Δt_2 and Δt_3 are too short. Assumptions to calculate these periods might be conservative. Based on the results, if an intended service period for the given required performance, *i.e.* a crack width is below 0.8 mm is longer than 13.7 years then materials properties and cover depth and so on need to be changed to meet the requirement. Also, a repair strategy and a protection methodology such as surface coating must be planed.

4. Conclusion

A performance-based evaluation for a reinforced concrete beam in a given chloride-laden environment was introduced. A sample calculation was conducted where a required performance was examined on a rational basis. The concept of the examination will be included in a future design methodology for concrete structures subjected to chloride ingress. However, uncertainties in each mechanisms involved and analytical methods may need to be justified with some safety factors. Future works will be directed to increase the reliability of the examination.

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A Proposal of Design System for Strengthening of Existing Concrete Structures by Performance-Based Design

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Summary

The paper presents the outline of the new design system for retrofitting of existing concrete structures on the basis of the performance-based design. Concrete structures, which are retrofitted with external cable, continuous fiber materials, steel plates, and concrete, are systematically verified if they have sufficient performance to satisfy required level with respect to all required performance items including structural safety and serviceability. Time-dependent performance deterioration of structures during service life is taken into account in the process of performance verification, so that durability and structural design are reasonably integrated.

1. Introduction

The Sub-Committee 307 under the Concrete Committee in the Japan Society of Civil Engineers has made the draft of the new design system for retrofitting of existing concrete structures in 1998[1][2]. The design system is on the basis of the concept of the performance-based design, which is accepted as the suitable design concept for general concrete structures in the next generation. The proposed design system consists of two parts: the basic frame and the retrofitting design manual. The former part contains the basic concept and common descriptions, while the latter does recommended equations to verify performances of structures retrofitted with external cable, continuous fiber materials, steel plates, and concrete. This paper focuses on outlining the former part, i.e. the basic frame of the proposed design system.

2. Performance of structures

Since the design system is based on the performance verification of structures, we preliminarily consider kinds of required performance of structures. Performance of structures are classified into following major categories.

- **Social and environmental friendliness**: ability to contribute to healthy social, economical, and cultural activities and to minimize harmful effects on surrounding social and natural environment.
- **Structural safety**: ability to avoid casualty due to structural failure and collapse.
- **Serviceability**: ability to make users of and people around the structures feel comfortable with the structures and to provide functions such as water tightness.
- **Constructability**: ability to assure safety and reliability during construction.
- **Easy Maintenance**: ability to make routine maintenance easy and to restore lessened performances in an economically and technically feasible way.
- **Easy demolition and recycling**: ability to make demolition and recycling easy.

Several performance items are contained in the above major categories as shown in Table 1. For each structure, a set of performance items is required quantitatively depending on kind, usage, and importance of the structure. In the proposed system, performance of structure is evaluated as a function of time, considering time-dependent deterioration due to loading and environmental attack. This is why durability of structures is not listed in the above categories. Durability can be implicitly considered by evaluating all other performance items along the time.

Table 1 Performance items

Categories	Items	
Social and environmental friendliness	Contribution to social, economical, and cultural activities	
	Harmful effect on surrounding social and natural environment	
Structural safety	Failure and collapse (normal action)	
	Failure and collapse (seismic action)	
	Stability	
Serviceability	Comfortable feeling	Comfortable ride / walk
		Anti-vibration
		Anti-noise
		Soundproof
		Heat-insulation
		Anti-odor / humidity
		Aesthetics
		Visual safety
	Functioning	Water-tightness
		Air-tightness
		Energy and substance insulation
Constructability	Safety	Danger to workers / surroundings
	Reliability	Quality of materials in the structure
Easy maintenance	Restorability	
	Easiness of routine inspection of the structure	
	Easiness of inspection of materials in the structure	
Easy demolition and recycling	Easy demolition	
	Easy disposal and recycling of materials	Easy disposal of materials
		Possibility of recycling of materials

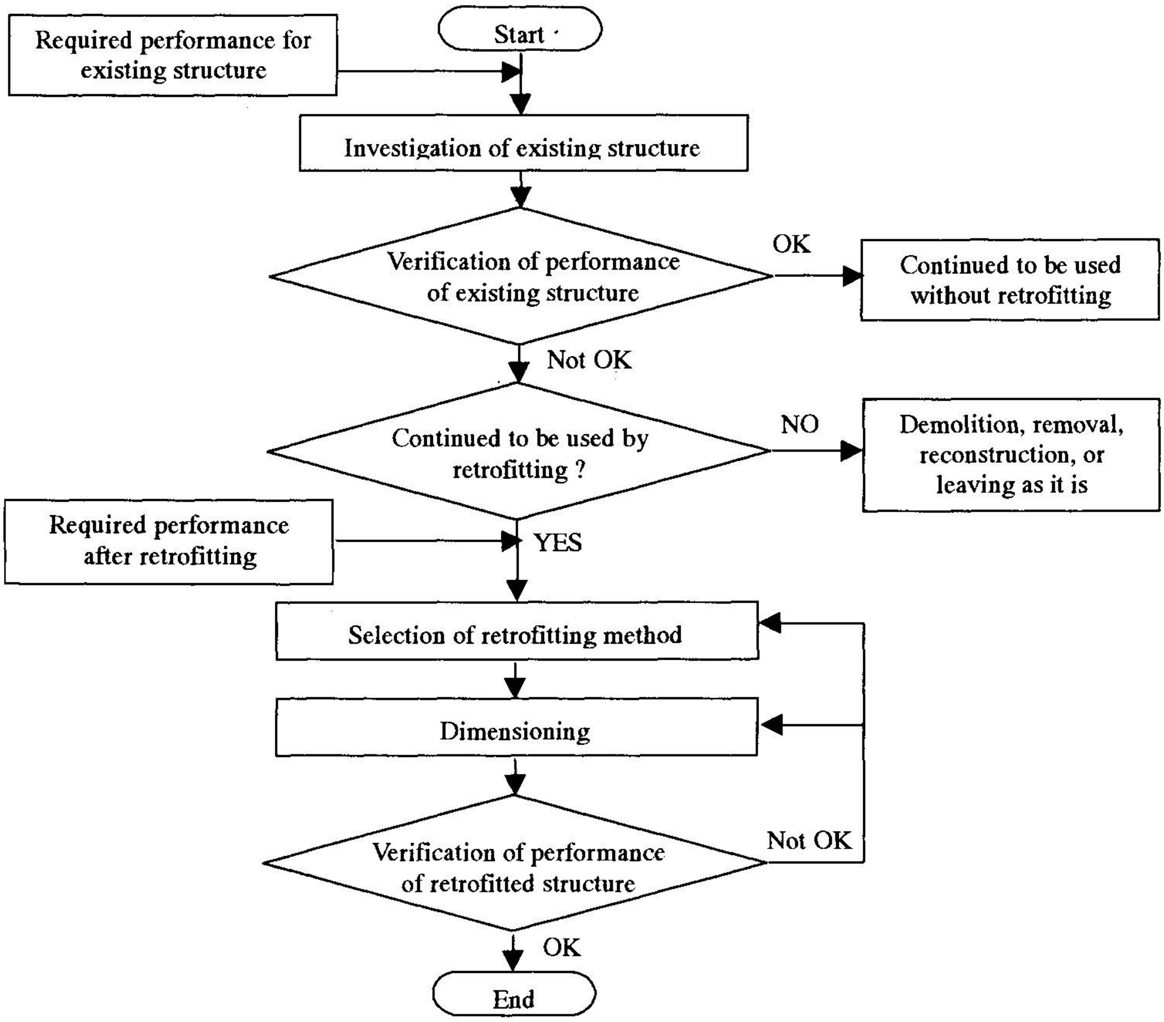

Fig.1 Flowchart of the proposed retrofitting design system

3. Outline of the proposed design system for retrofitting

The flowchart of the proposed retrofitting design system is shown in Fig.1. The system contains 1) investigation of the existing structure, 2) verification of performance of the existing structure, 3) selection of retrofitting method, and 4) verification of performance of the structure after being retrofitted. The objective existing structure is preliminarily investigated to obtain necessary information to evaluate its residual performances. Then, the residual performances of the structure at that moment are evaluated and verified if they satisfy the required level or not with respect to all required performance items. If it is found that the structure does not possess sufficient level of performance for some items and if the structure should be continuously used, then the retrofitting design proceeds according to the flowchart. An appropriate retrofitting method should be selected. Performances of the structure retrofitted by the selected method are evaluated and verified with required performances after retrofitting. In the performance verification of retrofitted structure, we have to confirm that the structure will keep sufficient level of performance to satisfy requirements at any time after retrofitting until the end of service life, considering time-dependent performance deterioration.

4. Basic concept of verification of performance

To practice performance verification, both performance of structures and requirement should be expressed quantitatively. Hence, each performance item listed in Table 1 should be represented by a corresponding physical variable which can be evaluated through available computational methods. This variable is called performance index in this study[3]. For instance, in order to verify structural safety, we conventionally quantify performance of structure in terms of axial force capacity, flexural capacity, shear capacity, and etc. If a highly reliable finite element program were available, structural safety could be verified by the computed structural behavior under the given loading action without any consideration of axial force capacity and etc. Consequently, performance index for each performance item depends on employed computational method. Table 2 shows examples of performance indices for some performance items, which are mainly related to mechanical characteristics of structures. Figure 2 shows the basic flowchart for verification of performance using performance index.

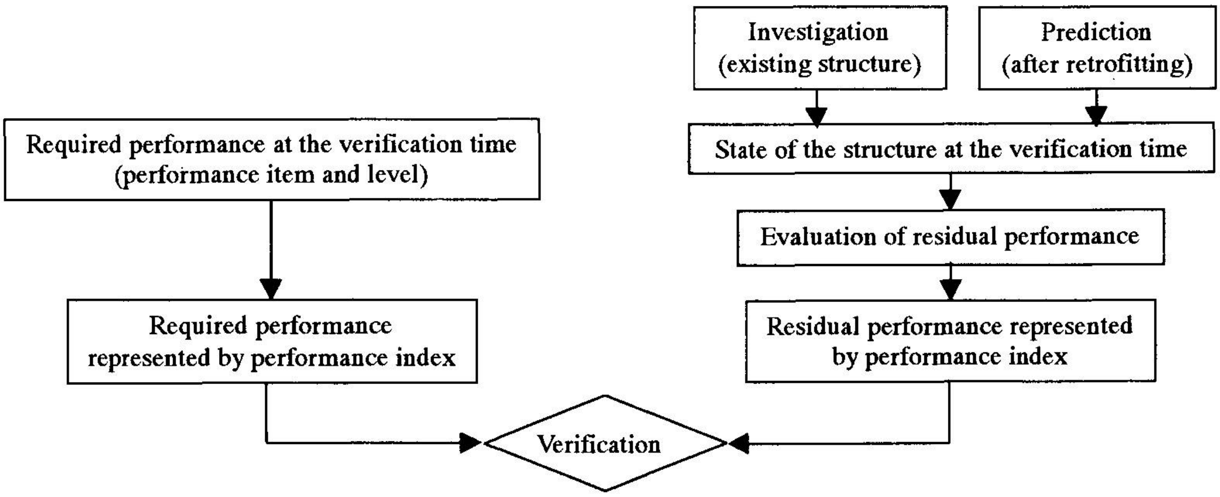

Fig.2 Verification of performance using performance index

Table 2 Performance indices

Categories	Items	Indices (by simple method)	Indices (by precise method)
Structural safety	Failure and collapse (normal action)	Axial force capacity, Flexural capacity, Shear force capacity, Torsional moment capacity, Fatigue strength, and Ultimate deformation	FEM analysis
	Failure and collapse (seismic action)		
	Stability	Overturning moment	
Serviceability	Comfortable ride / walk	Deflection, Step, Stiffness, Gap, Flatness and type of pavement	Acceleration transferred to passenger / walker
	Anti-vibration	Type of pavement, Stiffness, and Mass	Vibration level
	Anti-noise		Noise level
	Soundproof	Type of soundproof wall	
	Aesthetics	Crack width, Crack density, and Size and density of stain	
	Visual safety	Deflection, Crack width, and Crack density	
Easy maintenance	Restorability	Remaining deformation, Remaining crack width, and Damage in materials	FEM analysis

5. Verification of performance along the time

Retrofitted structures should have sufficient level of performance for all required performance items at any time after retrofitting until the end of their service lives. To confirm this, we should carry out performance verification along the time axis, considering time-dependent performance change due to loading and environmental actions. Figure 3 schematically shows the concept of performance verification of structure after retrofitting.

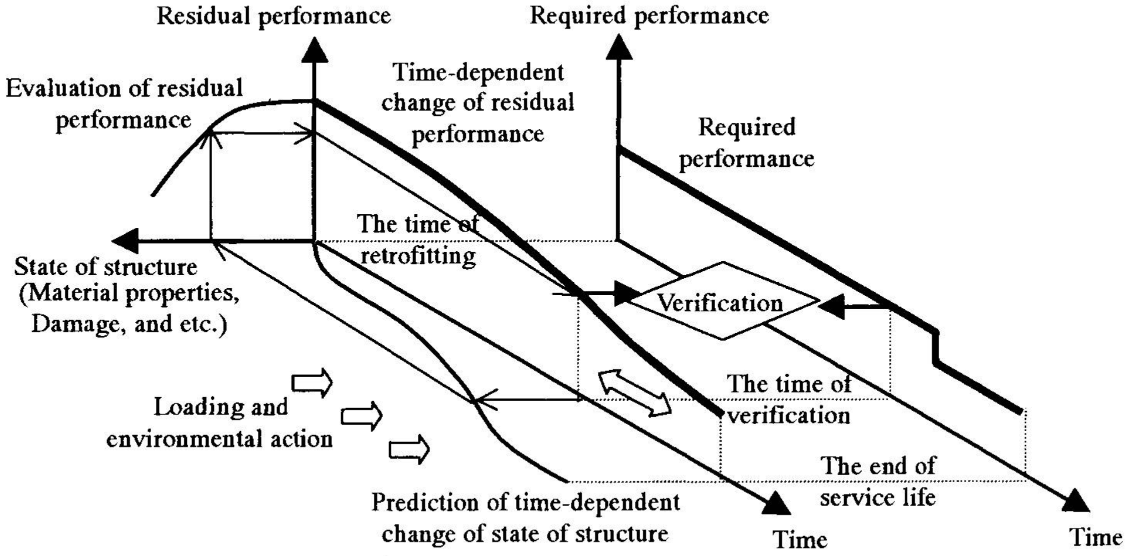

Fig.3 Verification of performance along the time

To attain the scheme shown in Fig.3, we need two essential technologies, which are 1) prediction of time-dependent change of state of structure and 2) evaluation of residual performance of structure at each time. The latter was discussed in the chapter 4. As for the former problem, we have to predict time-dependent change of state of structure, which includes material properties within structure, accumulated damage, concrete crack, corrosion of reinforcing bars, and remaining deformation, under given loading and environmental conditions. At the current stage of technology, however, we have not yet developed universal computational method that can precisely simulate every possible time-dependent change in concrete structures based on physical and chemical models. We should, therefore, estimate it by means of available simplified methods. At the simplest level of assumptions, deterioration of materials within structures may be accounted by material safety factors which are appropriately determined considering material types, structural types, construction conditions, and loading and environmental conditions. Proper detailing, such as minimum concrete cover, may further simplify the process to consider the time-dependent change in material properties. For example, if a thick concrete cover is provided, the corrosion of steel reinforcement may not be considered at all. Nevertheless, the proposal shown in Fig.3 would realize the integration of durability and structural design in the most rational way. The authors, therefore, insist that further study should be focussed on development of reliable prediction method for time-dependent change of structures to complete the proposed design system.

6. Conclusion

The outline of the design system for retrofitting of existing concrete structures which has following characteristics was presented in this paper:

1) The concept of the performance-based design is adopted.
2) Progress in concrete technology, such as development of precise simulation method, will be flexibly employed without any change of the framework of the design system.
3) Durability and structural design are rationally integrated taking time-dependent performance change into account in the process of performance verification.

Though not discussed in this paper, the JSCE Sub-Committee 307 has also proposed the recommended equations for evaluation of performance indices of structures retrofitted by major retrofitting methods, namely external cable method, wrapping or jacketing method using continuous fiber materials or steel plates, and concrete wrapping method. Major areas requiring more study to complete this design system are:

- Improvement of recommended equations for evaluation of performance indices,
- Development of universal prediction method for time-dependent change of structure due to loading and environmental attack.

Acknowledgment

The authors would like to express their sincere gratitude to the members of the JSCE Sub-Committee 307 who provided fruitful discussion for preparing this paper.

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Safety Evaluation System of Damaged Structures and Performance Based Design

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Summary

In the performance based design, it will be required to evaluate structural behaviors considering the deterioration of materials. We conducted an experiment to observe the effects of initial defects mainly due to environmental action upon structural behaviors. In our experiment, it was concluded that the main reason that affected the failure mode and ductility of RC beams was internal stress due to drying shrinkage. And then we proposed our future direction of establishing an analytical method to evaluate structural behaviors with material deterioration in time and space taken into account.

1. Introduction

The design systems of reinforced concrete structures are shifting to performance based design systems, which seems to be an international current. In the new systems, highly developed analytical methods are indispensable in order to check in a direct way whether the performances of structures surely satisfy required performances. Therefore, intensive researches have been done to develop analytical tools that can evaluate the structural performances of RC structures accurately. Conventionally, RC structures have been analyzed under realistic conditions that they are cured perfectly or material properties of them will never change. But it is apparent that in actual cases material properties of RC structures will change while they are in service just after construction, and that the performances of them will also change with time. Therefore, both structural and durability performances should be evaluated in an integrated way considering the changes of material properties in order to evaluate the lifetime performances of structures from birth to death. With this kind of tool, the performances of existing structures can be evaluated which seem to deteriorate compared to those of structures just after construction.

In this study, we discuss the necessity of evaluating the performances of structures with the deterioration of materials taken into account, and we suggest a future direction of establishing an evaluation tool for structures damaged mainly due to environmental action.

2. Damages of Structures due to Environmental Action

Real structures are destined to have internal stress introduced due to hydration heat, drying shrinkage, and autogenous shrinkage, and cracks might happen when the stress exceeds the tensile strength of concrete. These initial defects in concrete cause the large deterioration of resistance to mass transport, and substances harmful to reinforcing bars penetrate easily toward them through cracks, then under severe environment conditions reinforcing bars might be corroded. Therefore, initial defects have been considered to be one of the factors promoting corrosion of reinforcing bars or harmful to serviceability and appearance of structures.

But, it has not been discussed so much that initial defects might affect structural performance directly. As the shear capacity of concrete structures is affected so much by axial stress due to external force, it is likely enough that the capacity and the deformation of structure members are affected by internal stress. And it also seems possible that concrete cracks and internal stress distributed in structures may affect crack propagation when external force is applied. Therefore, in this study we concentrate on discussing the effects of initial defects mainly due to environmental action at early ages upon the structural behaviors, particularly, shear capacity and ductility of RC structures. Not so many experiments have been done in terms of structural behaviors considering initial defects, so we conducted fundamental experiments mentioned in the next chapter.

3. Experimental Study

3.1 Purpose

As mentioned above, the purpose of this experiment is to observe shear capacity and ductility under the presence of initial defects due to environmental action. The experiment was planned under a supposition that two RC beams made of the same material and cured in different conditions might fail in different modes. Because, compared to flexure capacity which depends on rather reinforcement than concrete properties, shear capacity seems to change so much due to the effect of initial defects upon crack propagation. Then, RC beams with no stirrups were provided and all of them were designed so that the flexure capacity was smaller than the shear capacity. And we observed the effect of internal stress and cracks generated by changing curing conditions. And, we used not only normal concrete but expansive concrete for the specimens in order to observe the effects of expansive agents. Furthermore, mechanically damaged specimens were provided in order to compare mechanical damage with initial defects. Therefore, the experiment consisted of 2 series. Series A for the specimens of different kinds of curing conditions, and Series B for the specimens damaged mechanically. The experiment was conducted in Asian Institute of Technology, Bangkok, Thailand.

3.2 Specimens

The dimensions of all the specimens are shown in the Fig.1. For Series A, 3 types of curing conditions were provided. One was water curing from 4 days after casting until just before testing, and another was water curing only for 7 days from 4 days after casting, and the other was being put in experiment room from 1 day after casting. Two kinds of concrete were used, that is, normal concrete and expansive concrete. Two specimens were provided for each material and each curing condition. Mix proportions of the specimens are shown in Table.1.

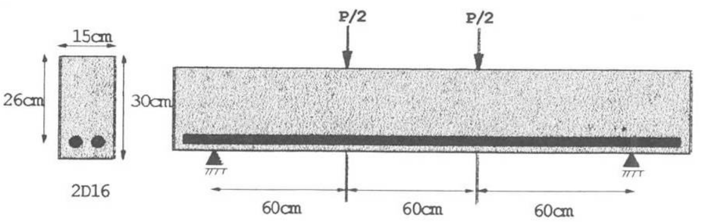

Fig.1 Geometry of the beams and Loading System

***Table.1** Mix proportion*

	Water [kg/m3]	Cement [kg/m3]	Expansive Agent [kg/m3]	Fine Aggregate [kg/m3]	Coarse Aggregate [kg/m3]	Superplasticizer [% by weight of cement]
Normal Concrete	176	375	0	805	948	0.25(%)
Expansive Concrete	176	337.5	37.5	805	948	0.275(%)

3.3 Loading System

Before testing, the compressive strength and the yielding stress of reinforcing bars were measured. Then the flexure capacity was calculated by JSCE code [1] formulation and the shear capacity was also calculated according to Okamura-Higai equation [2] shown as eq (1),

$$f_v = f_{v0}(0.75 + \frac{1.4}{a/d})(1 + \beta_p + \beta_d) \qquad (1)$$

$$f_{v0} = 0.94(f_c^{'})^{1/3}$$

$$\beta_p = \sqrt{100p} - 1$$

$$p = A_s/(bd)$$

$$\beta_d = \sqrt[4]{1/d} - 1$$

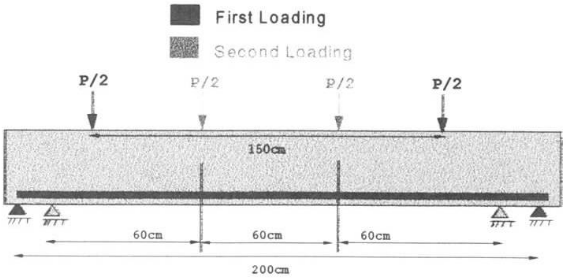

***Fig.2** Loading System for Series B*

where, f_v : shear stress when diagonal shear crack happens [kgf/cm^2], f_c' : compressive strength [kgf/cm^2], a : shear span [m], d : effective depth [m], b : width [m], A_s : total area of reinforcing bars [m^2].

According to the calculated results of flexure and shear capacity, the loading system was decided as shown in Fig.1.

The loading system applied for Series B was shown in the Fig.2. Bending cracks were supposed to be introduced in the first loading with wide loading span, and after unloading, the beams were subjected to the same loading system as the one in Fig.1.

3.4 Summary of the results

The summary of the results of Series A is given in Table.2. Shear and Flexure capacity calculated with measured compressive strength and yielding stress of reinforcing bar are also given in Table.2. Unfortunately, some of the cylinder specimens for compression test were not tested properly, so the measured values of them were excluded here.

***Table.2** Experimental Results (Series A)*

	Age at testing	fc'(kgf/cm2)	Shear Span (cm)	Curing (days)	Ultimate Load (tf)	Yielding Load (tf)	Failure Mode	Shear Capacity (tf)	Flexure Capacity (tf)
1NC 7+ 1.	27	453	60	23	10.61	9.7	flexure	10.77	9.75
1NC 7+ 2	29		60	25	10.58	9.8	flexure		
1NC 7 1	27		60	7	10.15	9.57	shear		
1NC 7 2	28		60	7	10.87	10.41	shear after yielding		
1NC 0 1	26	411	75	0	8.03	7.38	flexure	9.81	7.77
1NC 0 2	28		60	0	9.51		shear		
1NCEA 7+ 1	27	509	60	23	10.84	9.9	flexure	11.2	9.8
1NCEA 7+ 2									
1NCEA 7 1	27	375	60	7	10.41		shear (strange)	10.11	9.67
1NCEA 7 2	27	464	60	7	11.15	9.81	flexure	10.86	9.76
1NCEA 0 1	26	301	60	0	10.95	10.03	shear after yielding	9.4	9.55
1NCEA 0 2	28	374	60	0	10.24	9.78	shear after yielding	10.1	9.67

The specimens were named according to the following rules. The first *1* means Series A. ***NC*** means normal concrete. ***NCEA*** means expansive concrete. *7+* means all-time water curing. *7* means water curing for seven days. ***0*** means no water curing. *1* or *2* at the last mean the identification of specimens in the same condition.

1NC01 was tested first of all to decide the loading system. Therefore, only 1NC01 was tested under different loading system. Looking over the results of normal concrete, it can be said that failure mode may change according to curing conditions. As can be seen in Fig.3, normal concrete specimens under 3 types of curing conditions have completely different ductility. Of course it might be to some extent due to the difference of compressive strength. But the measured compressive strength did not show so much difference. And we should pay attention to the results of expansive concrete. When we replaced 10% of cement with expansive agents, ductility of all beams were improved so much as shown in Fig.4 though the compressive strength was almost the same as that of normal concrete.

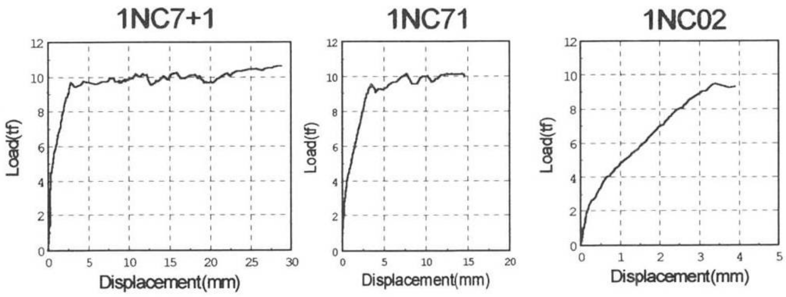

Fig.3 *Load-Displacement Relationship at the center of the span (Normal Concrete)*

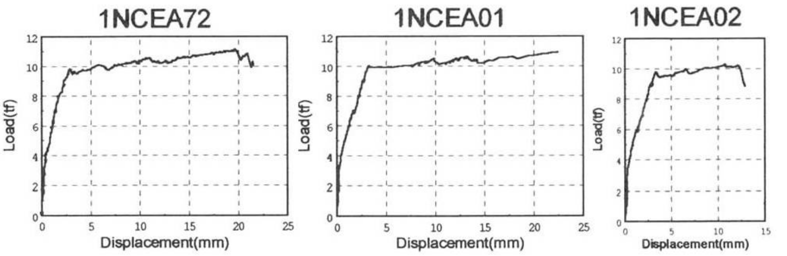

Fig.4 *Load-Displacement Relationship at the center of the span (Expansive Concrete)*

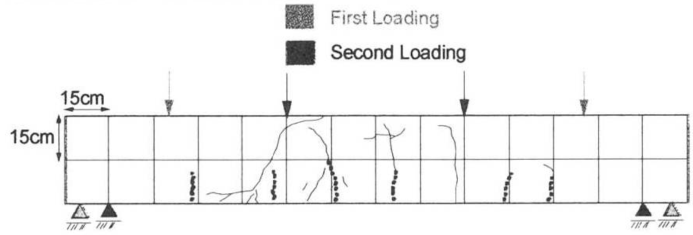

Fig.5 *Crack Pattern of Series B (Normal Concrete, Water Curing)*

The crack pattern of normal concrete beam in Series B is shown in Fig.5. At first loading 6 cracks were introduced with almost uniform crack spacing in as wide span as possible. After unloading, the beam was subjected to the same loading as Series A. Not so much difference was seen between normal concrete of Series A and that of B in terms of ultimate strength and ductility.

3.5 Discussions

Each couple of specimen on the same curing condition showed almost the same results except only 1NCEA71, so we are sure that on the whole the experiment was conducted accurately.1NCEA71 showed very brittle shear failure, which was completely different from 1NCEA72 that failed in flexure after enough deformation. Therefore we broke 1NCEA71 and checked the inside of it, but could not find any clear cause.

Concerning 1NC7+1, the shear capacity obtained from equation (1) was about 1.11 times larger than the flexure capacity obtained from JSCE code. It was clear that on these conditions, failure mode and ductility changed so much according to curing conditions. There seems to be many reasons for it, such as difference of compressive strength, internal stress, and cracking due to internal stress. Judging from the amount of powder used in this experiment, rather drying shrinkage than autogenous shrinkage was dominant. And the compressive strength of 1NC01 was not much smaller than 1NC7+1. Therefore it can be guessed that main reasons for changing of failure mode and ductility were internal stress and cracks.

In general, expansive agent replaced with part of cement compensate drying shrinkage, and large amount of expansive agent introduces chemical prestress in concrete. In this experiment, compressive strengths of normal concrete and expansive concrete were almost the same, so it seems reasonable that structural performances of expansive concrete were greatly improved because there were no internal tensile stress and cracks.

In Series B, cracks introduced mechanically did not affect the failure mode and ductility. Comparison between the results of Series A and those of Series B indicates that dominant reason for changing of structural performances is not cracking but internal stress distributed inside the specimen.

In actual RC structures shear reinforcement is arranged, therefore internal stress will hardly affect the capacity of them. But it is likely that the effects of internal stress should be considered in terms of ductility.

4. Analytical Evaluation Method (Our Future Direction)

As we have mentioned in the introduction, the performance based design obligates quantitative assessment of required performances by means of transparent and objective science. Therefore, a 3D finite element computer code named **COM3** for structural dynamics has been developed at the University of Tokyo for dynamic as well as static ultimate limit states. It can simulate structural behaviors expressed by displacement, deformation, stresses and macro-defects of materials in view of continuum plasticity, fracturing and cracking. Smeared crack modeling for reinforced concrete has been developed in the past decade at the University of Tokyo. The constitutive law can handle multi-directional cracking in 3D space by decomposing stress bearing mechanism into orthogonal 2D sub-planes on which 2D in-plane constitutive model of reinforced or plain concrete is applied [3][4].

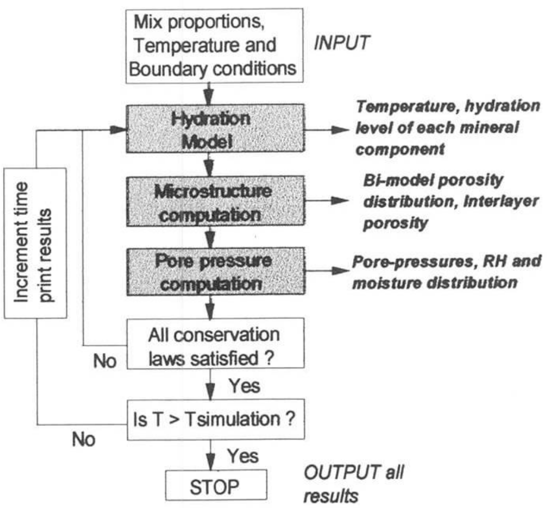

***Fig.5** Framework of **DuCOM** thermo-physics*

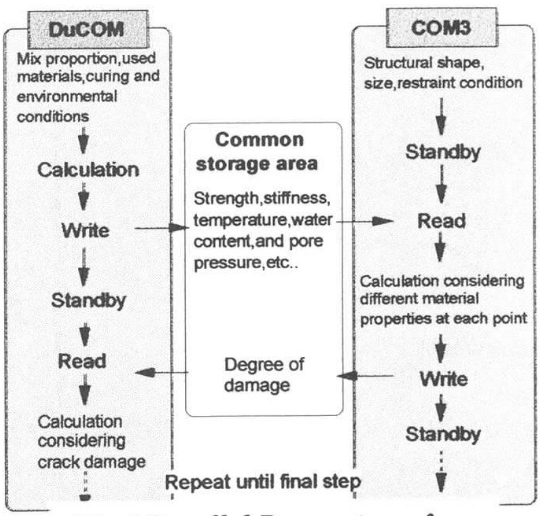

Fig.6** Parallel Processing of **DuCOM and COM3

In **COM3**, the change of material properties with time is not taken into account. Therefore, when we try to simulate the experimental results in this study, we cannot trace the phenomena faithfully but we have to depend on some simplified methods, that is to say, adjusting tensile strength or applying axial force in tension or compression. The structures with simple reinforcing arrangement used in this study could be analyzed with those simplified methods. But it is likely that real structures with shear reinforcement and with much more complicated shape have to be analyzed considering different material properties from part to part and internal stress distributed inside the structures.

Independent of **COM3**, a computational tool has been developed to trace properties of concrete for arbitrary materials, mix proportions, curing conditions, and dimensions. The hydration of cement in concrete volume, moisture migration, and micro-pore structural formation exhibit strong mutual inter-link. 3D finite element analysis program named **DuCOM** for simulating these transient phenomena in space and time has been developed at the University of Tokyo. The framework of **DuCOM** is shown in Fig.5 and the details were presented in literatures [5][6][7][8].

Furthermore, by solving two systems in parallel, that is, **COM3** and **DuCOM**, we already have a unified approach of mechanics which governs stress and strain fields and thermo-hygro physics [8]. Parallel processing of **DuCOM** and **COM3** is shown in Fig.6. With this kind of tool, we can evaluate the structural behaviors in time and space considering the development or deterioration of constitutive materials of structures. Though we are now in an immature stage, in the near future it will be possible to deal with problems like the experiment in this study or more complicated matters.

5. Conclusions

In the performance based design, it will be required to evaluate structural performances with the change of material properties with time taken into account. In this study, in order to observe the effects of damages due to environmental action upon structural behaviors, 2 series of experiments were conducted. From the results, it was concluded that in some conditions structural behaviors such as failure mode and ductility are affected by curing conditions. Internal stress mainly due to drying shrinkage was considered to be the main reason that caused the change of structural behaviors. In order to deal with this kind of problem, that is, structural behaviors with the change of material properties, we proposed our future direction of establishing analytical evaluation method, that is, a unified approach of mechanics which governs stress and strain fields and thermo-hygro physics.

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Analysis of Cracking Localization Using the Smeared Crack Approach

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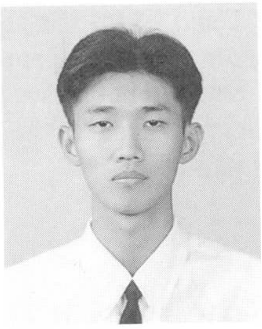

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Summary

In order to consider cracking localization in the finite element analysis, it is more suitable to have an energy expression written in terms of discrete irreversible variables, which will allow differentiation of the energy expression with respect to those irreversible variables. This implies that the discrete crack approach should be more appropriate to this kind of analysis than the smeared crack approach. However, the discrete crack approach in the finite element analysis may not be the best solution for problems with many cracks, which are unavoidable for the analysis of cracking localization. To avoid the drawbacks in both approaches, a special treatment on the smeared crack finite element analysis to allow the consideration of cracking localization is proposed. In the method, a discrete irreversible variable related to crack strain is introduced, and cracking localization is investigated, based on this discrete irreversible variable.

1. Introduction

It is commonly known that cracking localization prior to the failure plays a very important role in the fracture behavior of quasi-brittle materials such as concrete. In order to capture the ultimate capacity of such materials in structures, it is necessary to consider not only formation and propagation of cracks but also localization of them. In the analysis of cracking localization, consideration of stability and bifurcation of equilibrium states is one of the tasks that have to be done. To this end, the stationary condition of the energy of the system can be examined. This requires expression of the energy in terms of discrete irreversible variables. Having the energy written as a function of discrete irreversible variables allows differentiation of the energy expression with respect to these irreversible variables, which is necessary for the consideration of the stability condition. This fact implies that the discrete crack approach in the finite element method may be more suitable for the cracking localization analysis than the smeared crack approach. In the discrete crack approach, the irreversible variables that have to be considered are the crack opening displacements. These crack opening displacement variables are usually discretized along crack paths and treated as the degrees of freedom in the analysis. Therefore, the energy of the system can be expressed in terms of these degrees of freedom. Computing the first and second variations of the energy with respect to the crack opening displacement degrees of freedom can be done easily. On the contrary, if the smeared crack approach is employed, the energy of the system will be expressed in terms of irreversible crack strain components. These crack strain components are functions of position. To compute the first and second variations of the energy with respect to these crack strain functions, complex mathematics involving the calculus of variations must be employed.

Nevertheless, the discrete crack approach may not perform best when there are many cracks. In the cracking localization analysis, there will be many cracks in the domain. Having many cracks in the domain leads to more degrees of freedom, and the mesh topology of the problems may have to be changed drastically. Moreover, the singularity problem of the system stiffness equation may also appear. These problems can be mostly avoided if the smeared crack approach is employed. In the smeared crack model, no increase in the degrees of freedom or change in the mesh topology is required. Although the smeared crack model may also face the singularity problem of the system in case of softening materials, the problem is less serious than that of the discrete crack model.

In this study, a special consideration on the smeared crack finite element analysis is proposed. The proposed consideration will make it possible to consider cracking localization even when the smeared crack model is used. In the proposed method, a discrete irreversible variable related to the crack strain is introduced in the smeared crack model. This discrete variable will allow the consideration of stability and bifurcation of the equilibrated solution to be done easily by considering the variations of the energy with respect to the proposed discrete variable.

2. Smeared Crack Finite Element Analysis for Cracking Localization

The fundamental scheme of the smeared crack model is the decomposition of the total strain increment $\Delta\varepsilon$ into an elastic strain increment $\Delta\varepsilon_e$ and a crack strain increment $\Delta\varepsilon_{cr}$, i.e.,

$$\Delta\varepsilon = \Delta\varepsilon_e + \Delta\varepsilon_{cr}. \tag{1}$$

Therefore, the total energy increment can be written as

$$\begin{aligned}\Delta U &= \Delta U^m + \Delta U^d \\ &= \left[\frac{1}{2}\int_V \Delta\varepsilon_e^T \mathbf{D}_e \Delta\varepsilon_e dV - \int_S \Delta\mathbf{u}^T \Delta\mathbf{t} dS - \int_V \Delta\mathbf{u}^T \Delta\mathbf{f} dV\right] + \left[\frac{1}{2}\int_V \Delta\varepsilon_{cr}^T \hat{\mathbf{D}}_{cr} \Delta\varepsilon_{cr} dV\right]\end{aligned} \tag{2}$$

where ΔU^m and ΔU^d represent the mechanical potential energy increment and the dissipated energy increment, respectively [1, 2]. Here, $\Delta\mathbf{t}$ and $\Delta\mathbf{f}$ denote the surface traction increment vector and the body force increment vector, respectively. In addition, $\mathbf{D}_e$ denotes the elastic constitutive matrix for the uncracked solid, i.e.,

$$\sigma = \mathbf{D}_e \Delta\varepsilon_e, \tag{3}$$

and $\hat{\mathbf{D}}_{cr}$ is a matrix defined as

$$\hat{\mathbf{D}}_{cr} = \hat{\mathbf{N}}^T \mathbf{D}_{cr} \hat{\mathbf{N}} \tag{4}$$

where $\mathbf{D}_{cr}$ is the constitutive matrix defining the relation between the crack strain increment $\Delta\mathbf{e}_{cr}$ and the crack stress increment $\Delta\mathbf{s}_{cr}$ in the crack local coordinate system [3], i.e.,

$$\Delta\mathbf{s}_{cr} = \mathbf{D}_{cr} \Delta\mathbf{e}_{cr}, \tag{5}$$

and $\hat{\mathbf{N}}$ is the transformation matrix defined as

$$\Delta\mathbf{e}_{cr} = \hat{\mathbf{N}} \Delta\varepsilon_{cr}. \tag{6}$$

In the expression for the total energy increment in Eq. (2), the irreversible variable that has to be considered in the stability analysis is the crack strain increment $\Delta\varepsilon_{cr}$. The first and second variations of the total energy increment with respect to this crack strain increment must be obtained in order to get the equilibrium path and the stability condition of the equilibrium path. Since the total energy increment is a functional of the crack strain increment function, the calculus of variations is required. To avoid this difficulty, we introduce a crack displacement increment vector $\Delta\mathbf{u}_{cr}$ defined as

$$\Delta\mathbf{u} = \Delta\mathbf{u}_e + \Delta\mathbf{u}_{cr} \tag{7}$$

where $\Delta\mathbf{u}$ and $\Delta\mathbf{u}_e$ are the total displacement increment vector and the displacement increment vector corresponding to the elastic strain, respectively.

Consider the i^{th} element in the finite element analysis. The element is assumed to be a cracked element. Interpolate these three displacement increments from nodal quantities, i.e.,

$$\Delta\mathbf{u} = \mathbf{N}\Delta\mathbf{U}, \quad \Delta\mathbf{u}_e = \mathbf{N}\Delta\mathbf{U}_e, \quad \Delta\mathbf{u}_{cr} = \mathbf{N}\Delta^i\mathbf{U}_{cr}, \quad \Delta\mathbf{U} = \Delta\mathbf{U}_e + \Delta^i\mathbf{U}_{cr} \tag{8}$$

where $\Delta\mathbf{U}$ is the nodal total displacement increment vector, $\Delta\mathbf{U}_e$ is the nodal displacement increment vector corresponding to the elastic strain, and $\Delta^i\mathbf{U}_{cr}$ is the nodal crack displacement increment vector. Here, $\mathbf{N}$ is the shape function matrix. Note that the superscript i for the i^{th} element is necessary for the nodal crack displacement increment vector $\Delta^i\mathbf{U}_{cr}$ because the nodal crack displacement increments of the same node for different elements can be different. This is natural because, in the smeared crack approach, cracking in each element is completely independent of each other.

Computing strains from Eq. (7), we obtain Eq. (1), i.e.,

$$\Delta\varepsilon = \Delta\varepsilon_e + \Delta^i\varepsilon_{cr} \tag{9}$$

where

$$\Delta\varepsilon = \mathbf{B}\Delta\mathbf{U}\,,\ \Delta\varepsilon_e = \mathbf{B}\Delta\mathbf{U}_e,\ \text{and}\ \Delta^i\varepsilon_{cr} = \mathbf{B}\Delta^i\mathbf{U}_{cr} \tag{10}$$

in which **B** is the strain-displacement matrix.

Rewrite Eq. (2) as

$$\Delta U = \frac{1}{2}\int_V \left(\Delta\varepsilon - \Delta^i\varepsilon_{cr}\right)^T \mathbf{D}_e \left(\Delta\varepsilon - \Delta^i\varepsilon_{cr}\right) dV + \frac{1}{2}\int_V \Delta^i\varepsilon_{cr}^T \hat{\mathbf{D}}_{cr} \Delta^i\varepsilon_{cr} dV - \int_V \Delta\mathbf{u}^T \Delta\mathbf{f} dV - \int_S \Delta\mathbf{u}^T \Delta\mathbf{t} dS \tag{11}$$

which yields

$$\begin{aligned}\Delta U = {} & \frac{1}{2}\int_V \Delta\varepsilon^T \mathbf{D}_e \Delta\varepsilon dV - \frac{1}{2}\int_V \Delta\varepsilon^T \mathbf{D}_e \Delta^i\varepsilon_{cr} dV - \frac{1}{2}\int_V \Delta^i\varepsilon_{cr}^T \mathbf{D}_e \Delta\varepsilon dV + \\ & \frac{1}{2}\int_V \Delta^i\varepsilon_{cr}^T \mathbf{D}_e \Delta^i\varepsilon_{cr} dV + \frac{1}{2}\int_V \Delta^i\varepsilon_{cr}^T \hat{\mathbf{D}}_{cr} \Delta^i\varepsilon_{cr} dV - \int_V \Delta\mathbf{u}^T \Delta\mathbf{f} dV - \int_S \Delta\mathbf{u}^T \Delta\mathbf{t} dS.\end{aligned} \tag{12}$$

Substituting Eqs. (8) and (10) into Eq. (12), and applying the stationary condition $\delta(\Delta U) = 0$ give

$$\begin{bmatrix} \int_V \mathbf{B}^T \mathbf{D}_e \mathbf{B} dV & -\int_V \mathbf{B}^T \mathbf{D}_e \mathbf{B} dV \\ -\int_V \mathbf{B}^T \mathbf{D}_e \mathbf{B} dV & \int_V \mathbf{B}^T \mathbf{D}_e \mathbf{B} dV + \int_V \mathbf{B}^T \hat{\mathbf{D}}_{cr} \mathbf{B} dV \end{bmatrix} \begin{Bmatrix} \Delta\mathbf{U} \\ \Delta^i\mathbf{U}_{cr} \end{Bmatrix} = \begin{Bmatrix} \int_V \mathbf{N}^T \Delta\mathbf{f} dV + \int_S \mathbf{N}^T \Delta\mathbf{t} dS \\ \mathbf{0} \end{Bmatrix} \tag{13}$$

which is the element stiffness equation for the i^{th} element. After assembling all element stiffness equations and applying prescribed displacements and forces, the static condensation can be used to remove the nodal total displacement increment from the obtained global matrix equation. Therefore, the equation can be written in the following form, i.e.,

$$\mathbf{K}_{cr}\Delta\mathbf{U}_{cr} = \Delta\mathbf{R}_{cr} \tag{14}$$

where $\mathbf{K}_{cr}$ and $\Delta\mathbf{R}_{cr}$ are the stiffness matrix and the right-hand-side force increment vector after the static condensation.

It must be noted that Eq. (14) is a singular equation because $\Delta\mathbf{U}_{cr}$ contains the rigid-body crack displacements. To avoid these rigid-body crack displacements, constraints to remove them must be applied to the equation (see the next section). This leads to a modified equation, i.e.,

$$\hat{\mathbf{K}}_{cr}\Delta\hat{\mathbf{U}}_{cr} = \Delta\hat{\mathbf{R}}_{cr}. \tag{15}$$

The stability condition can be obtained by checking the eigenvalues of $\hat{\mathbf{K}}_{cr}$. If all the eigenvalues are positive, it means the equilibrium is stable and there is no bifurcation.

3. Results and Discussion

To show the validity of the method, a simple one-dimensional cracking localization problem is considered. The problem is shown in Fig. 1. It is an axial bar with one fixed support. At the other end, the displacement is controlled. The total length of the bar is $4L$ and the area is A. The material is assumed to be elastic with Young's modulus and Poisson's ratio equal to E and 0, respectively. The bar is discretized into 4 axial two-noded elements, each of which has the length of L. Assume that there are two existing cracks in the elements ***2*** and ***3***. Further assume that the ratio between the incremental transmitted axial stress and the incremental axial crack strain, $\Delta\sigma / \Delta\varepsilon_{cr}$, for the current incremental step is equal to H for both cracks.

The linear shape function is used for all elements. Therefore, for the elastic bar elements without cracks ***1*** and ***4***, the element stiffness equations will be the normal ones. However, for the cracked elements ***2*** and ***3***, the stiffness equations are written as

$$\begin{bmatrix} \frac{AE}{L} & -\frac{AE}{L} & -\frac{AE}{L} & \frac{AE}{L} \\ -\frac{AE}{L} & \frac{AE}{L} & \frac{AE}{L} & -\frac{AE}{L} \\ -\frac{AE}{L} & \frac{AE}{L} & \frac{A(E+H)}{L} & -\frac{A(E+H)}{L} \\ \frac{AE}{L} & -\frac{AE}{L} & -\frac{A(E+H)}{L} & \frac{A(E+H)}{L} \end{bmatrix} \begin{bmatrix} \Delta U^j \\ \Delta U^k \\ \Delta^i U^j_{cr} \\ \Delta^i U^k_{cr} \end{bmatrix} = \begin{bmatrix} \Delta^i R^j \\ \Delta^i R^k \\ 0 \\ 0 \end{bmatrix} \tag{16}$$

where $i = 2, j = 2, k = 3$ for element ***2*** and $i = 3, j = 3, k = 4$ for element ***3***. The superscript before a variable denotes the element number while the one after denotes the node number.

Assembling all the element stiffness equations, applying the prescribed conditions and removing the total displacement increment degrees of freedom by the static condensation give

$$\begin{bmatrix} \frac{A(E+4H)}{4L} & -\frac{A(E+4H)}{4L} & \frac{AE}{4L} & -\frac{AE}{4L} \\ -\frac{A(E+4H)}{4L} & \frac{A(E+4H)}{4L} & -\frac{AE}{4L} & \frac{AE}{4L} \\ \frac{AE}{4L} & -\frac{AE}{4L} & \frac{A(E+4H)}{4L} & -\frac{A(E+4H)}{4L} \\ -\frac{AE}{4L} & \frac{AE}{4L} & -\frac{A(E+4H)}{4L} & \frac{A(E+4H)}{4L} \end{bmatrix} \begin{bmatrix} \Delta^2 U^2_{cr} \\ \Delta^2 U^3_{cr} \\ \Delta^3 U^3_{cr} \\ \Delta^3 U^4_{cr} \end{bmatrix} = \begin{bmatrix} -\frac{AE}{4L}\Delta\bar{u} \\ \frac{AE}{4L}\Delta\bar{u} \\ -\frac{AE}{4L}\Delta\bar{u} \\ \frac{AE}{4L}\Delta\bar{u} \end{bmatrix} \tag{17}$$

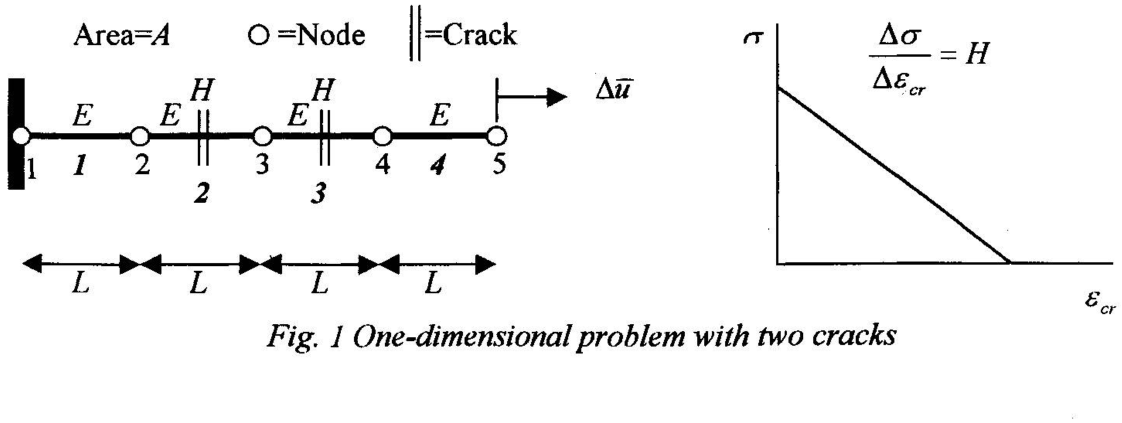

Fig. 1 One-dimensional problem with two cracks

As discussed earlier, the equation obtained above is singular due to the rigid-body crack displacements. To constrain the rigid-body crack displacement, the crack displacements at the centers of the cracked elements are set to zero. From the linear interpolation, we get

$$\Delta^2 u_{cr}(0) = \frac{1}{2}\Delta^2 U_{cr}^2 + \frac{1}{2}\Delta^2 U_{cr}^3 = 0, \; \Delta^3 u_{cr}(0) = \frac{1}{2}\Delta^3 U_{cr}^3 + \frac{1}{2}\Delta^3 U_{cr}^4 = 0. \tag{18}$$

Using Eqs. (18) in Eq. (17) leads to

$$\begin{bmatrix} \frac{A(E+4H)}{2L} & -\frac{AE}{2L} \\ -\frac{AE}{2L} & \frac{A(E+4H)}{2L} \end{bmatrix} \begin{bmatrix} \Delta^2 U_{cr}^2 \\ \Delta^3 U_{cr}^4 \end{bmatrix} = \begin{bmatrix} -\frac{AE}{4L}\Delta\bar{u} \\ \frac{AE}{4L}\Delta\bar{u} \end{bmatrix}. \tag{19}$$

The eigenvalues of the stiffness in Eq. (19) are used to obtain the stability of the equilibrium state. The eigenvalues are equal to $2AH/L$ and $A(E+2H)/L$. If $H>0$, two eigenvalues are positive and the equilibrium state is stable. This solution represents cases where hardening behavior occurs after cracking. This kind of hardening behavior is unrealistic for quasi-brittle materials. In this case, both cracks will open and there will be no localization. If $-E/2<H<0$, one eigenvalue is negative and the equilibrium state is unstable. This solution represents cases where softening behavior occurs after cracking. This softening behavior is common for quasi-brittle materials. The result means that the solution with two opening cracks is unstable. Actually, with further investigation, it can be shown that only one crack will open and the other crack will close. This behavior represents the localization. If $H<-E/2$, both eigenvalues are negative. This also corresponds to unstable equilibrium where both cracks will open in an unstable manner. These results agree with those obtained by Brocca [1] who investigated the same problem analytically.

4. Conclusion

The smeared crack approach can be used in the analysis of cracking localization by introducing an appropriate discrete irreversible variable related to the crack strain. In this study, a crack displacement variable is introduced in the smeared crack finite element analysis for this purpose. The relationship between the proposed variable and the crack strain follows the ordinary strain-displacement relationship. This newly introduced variable allows the consideration of stability of equilibrium states and bifurcation. The scheme is tried with a simple axial problem. The results obtained show promising capability of the method in analyzing problems with cracking localization. Therefore, using the discrete crack finite element, which is not suitable for problems with many cracks, in cracking localization problems can be avoided.

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Importance of Workmanship in Concrete Construction

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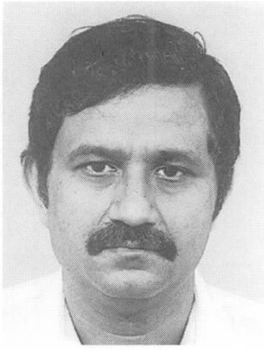

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Summary

The paper discusses the importance of workmanship in concrete construction and in particular focuses attention on Part II (Materials & Construction) of the Draft Concrete Model Code, where a sub-section on workmanship is devoted to all major aspects of concrete construction including formwork, reinforcement, materials, mixing, placing and curing of concrete.

To ensure good workmanship, it is recommended that dimensional tolerances are specified for all structural elements and these are checked for compliance during different phases of construction. The paper gives some tolerance limits commonly accepted in Australia for different types of structural elements, reinforcing bars and in the mixing and placing of concrete. It is recommended that these be adopted and used in the Concrete Model Code.

1. Introduction

In any civil engineering construction project, dimensions will seldom, if ever, equal exactly to what is specified or shown on the drawings. The deviations from the specified dimensions result from a combination of several factors, but it is seen that these are generally related to poor workmanship and lack of quality control measures exercised on site. If the recommended materials are used in construction and proper inspection procedures are followed, then the dimensional variations in the members could be significantly reduced and better workmanship would result.

With the availability of several computer packages these days, design of most civil engineering structures can be done to a high degree of accuracy and designers' with their good understanding of material properties can specify the most appropriate material suited for a particular end use, but unfortunately, they do not have any control over workmanship, as it depends on the skills of the workers and the quality control measures exercised on site. An accurate and a precise design, coupled with the use of excellent building materials cannot be transformed into a good building structure, if the workmanship is not proper. Moreover, if the standard of workmanship is poor, finished structure, apart from presenting an unpleasant appearance is not likely to perform according to its expected behaviour. To overcome the problem of dimensional variability and workmanship, it is suggested that instead of only specifying the dimensions for structural elements, acceptable limits of tolerance for members should also specified in the structural drawings. In some countries, this practice is already in existence and relevant Codes (1) have been developed and are available for compliance by builders and contractors. However, it has been observed that the standard of workmanship and dimensional variations are not a real threat in most developed countries, because well documented inspection procedures are already in place and inspection in all phases of construction is carried out as a matter of routine. However, the situation in developing countries is different, where quality and workmanship are not given due regard in many cases.

2.0 Essential Requirements for Quality and Workmanship

2.1 Inspection and Supervision

To achieve a high standard of workmanship in concrete construction, it is essential that proper inspection and supervision procedures are followed in all phases. A proper and a timely inspection will ensure that all design requirements laid down in the specifications and on the structural drawings are achieved in construction. The inspection and supervision work entails regular checks on all phases of the work in progress, as well as on the completed portions of the structure, to assess their compliance to the specifications.

Apart from the dimensional and alignment checks, site inspection also includes an examination of the building materials and formwork. In addition, site supervisors are also expected to carry out regular checks on mixing, placing, compacting and curing of concrete to ensure that proper quality of concrete is maintained in all phases of construction.

To have a tighter control on the quality of construction, the site supervisor should have an authority to refuse the use of materials which do not conform to the specifications and ones which are likely to result in improper construction. He should have an authority to stop any work in progress which is not in accordance with the approved plans and specifications for the job. The supervisor should also have the power to order for the removal or repair of faulty construction or for any construction performed without an inspection.

2.2 Workmanship in Formwork

The accuracy which can be achieved in concrete construction depends on the skills and workmanship exercised in the fabrication of formwork, because fresh concrete does not have a shape or configuration of its own, but acquires the shape and size of the formwork, after it sets and hardens. Therefore, it is essential that formwork must be built to correct dimensions and shape. It is also essential that the formwork has sufficient rigidity to maintain its shape and dimensional integrity under different types of construction loads. In addition, it should be stable and must be strong enough to align large members in position.

To achieve a proper shape, size and alignment of concrete elements, it is necessary that dimensions such as width, height and length of the formwork are calculated precisely, so that the dimensions of the finished member are within the acceptable limits of tolerance. It should be noted that no dimensional tolerance limits are specified in the Draft Concrete Model Code and to achieve any reasonable standard of construction in most developing countries, it will be more than desirable to have such limits. Some of the ACI Committee 347 recommended limits of tolerance for different structural elements as referred to in (2) are given in Table 1. A similar set of tolerance limits are also available for the footings of columns, piers, walls, buttresses and other structural members. In Australia, dimensional tolerances and permissible surface tolerances for surface finishes are specified in AS 1509 (3) and AS 1510, Part 1, (4) respectively. Similar, tolerance limits specified in the United Kingdom are given in BS 3626 (5).

Table 1

Acceptable tolerances for various structural elements

Structure	Tolerance
1. Variation in cross-section dimensions of columns, beams, buttresses, piers and similar members	- 0.25 in (6mm) + 0.5 in (12mm)
2. Variation in the thickness of slabs, walls, arch sections, and similar members	-0.25 in (6mm) +0.5 in (12mm)
3. Variations of dimensions to individual structure	1.25 in (32mm) in 80ft or more features from established positions (twice this amount for buried construction
4. Variation from the plumb from the specified batter or from the curved surfaces of all structure, including lines and surfaces of columns, walls, piers, buttresses, arch sections, vertical joint grooves and visible arrisses	0.5 in. (12mm) in 10 ft 0.75in. (19mm) in 20 ft 1.25in. (31mm) in 40 ft or more (twice above amounts for buried construction)
5. Variation from the level or from the grades indicated on the drawings in slabs, beams, soffits, horizontal joint grooves and visible arrises	0.25 in. (6mm) in 10 ft 0.50 in. (12mm) in 30 ft or more (twice above amounts for buried construction).

A comparison of the dimensional tolerances acceptable in the US (2) , Australia (3) and the U.K. (5) indicates only a marginal difference in some structural elements and in most cases, the values are identical. However, it is to be noted that no acceptable limits are not discretely specified in many Asian countries and judgement of the site engineers is usually relied upon. However, some Asian countries do mention in the contract documents that dimensions of the finished members should be within the tolerance limits given in the British Standard (5).

It is to be noted that fabrication of an excellent and a precise formwork, will also not necessarily ensure good surface finish and workmanship because, small errors in the assembly of formwork have been seen to lead to catastrophic failures. In some cases, timber formwork have failed because adequate number of nails were not provided during the assembly of the components while in other cases, proper tightening of nuts and locking devices was not carried out. Some of these problems can be overcome, if proper inspection and safety checks are carried out on site as a matter of routine.

2.3 Inspection of Reinforcement Bars before Concreting

To ensure proper quality of concrete construction, it is recommended in the draft Concrete Model Code that the surface of the steel bars should be free from mud, oil or grease or non-metallic coating and loose rust. These appear to be an ideal set of conditions for reinforcement bars and ones which are difficult to achieve in practice because, bars normally have a coat of natural rust on them. Moreover, steel is generally stored on site and close to the excavated ground and therefore, bars invariably pick up rust and are often covered with mud as well. A more realistic clause in the Model Code should be: 'the site supervisor should ensure that loose rust and mud are brushed off from the reinforcement before placement and he should also check that rusting has not caused excessive pitting or loss of cross-section of the reinforcement'.

To ensure good workmanship, it is essential to check that the reinforcement is fixed as shown in the structural drawings and is supported by approved concrete, metal or other chairs. The maximum permissible tolerances on fixing reinforcement in structural elements in accordance with the Australian specifications AS 1480 (6) are given in Table 2.

Table 2

Maximum permissible tolerances on fixing reinforcement

Location of reinforcement or dimension to the end for which the tolerance applies	Tolerance
In any member, the tolerance measured in the direction of 'd' where the overall depth or thickness, D in the same direction is	
(a) up to 300 mm	± 5mm
(b) more than 300 mm and up to 450 mm	± 10mm
(c)more than 450 mm and up to 600 mm	± 15mm
(d)more than 600 mm	± 20 mm
At that end of a bar, or at the outside edge of a bent bar where the location is	
(a) controlled by concrete cover	± 15mm
(b) not controlled by concrete cover	± 50mm
The specified lateral location of any one bar of slab or wall reinforcement, in the plane of the reinforcement, or of any one fitment, in the direction of the specified spacing	± 0.25 times the specified spacing

In addition to the above tolerances, the Australian specification also gives both the minimum and the maximum distance between parallel reinforcement bars. The minimum distance between any two bars is specified to permit proper placement and compaction of concrete, while the maximum spacing is specified to ensure adequate performance of the reinforced concrete member.

For the UK, the BSI publication - BS 4466 (7) sets out preferred shapes and gives guidance on cutting and bending tolerances of reinforcing bars, while the British Code of Practice BS 8110 (8) contains recommendations for cover to bars and permissible deviations.

It is to noted that workmanship in the placing of reinforcement bars for a concrete member is judged from the uniformity of spacing between the bars and the care exercised in securing the bars in position to ensure that they do not displace when the concrete is poured and compacted. In addition, it is important to ensure that the bar placing tolerances do not add with the tolerances of the formwork to leave steel with insufficient cover. While inspecting the layout and fixing of the reinforcement in a formwork, it is also necessary to check and ensure that the minimum concrete cover requirements for different exposure conditions are met.

2.4 Workmanship in Concreting

Workmanship in concreting is judged from the condition and tolerances of the finished surface. A good surface finish is one which is free from surface cracking and which does not have an unsightly differences in texture and colour, especially in exposed work. All these qualities can be achieved in a concrete element, only if, proper quality control measures are taken in all operations of concrete construction such as batching, mixing, placing, compaction, finishing and curing.

To have uniformity in concrete from one batch to the other, it is necessary to ensure that materials used on site are procured from the same sources and specified proportions of constituents are maintained while for the mixing process, the mixing time has to be adjusted to ensure that ingredients get uniformly distributed within the mass of concrete. The uniformity requirements for a concrete mix and the extent of variations permitted in Australia as specified in the Australian Concrete Inspection Manual (9) and a set of recommended values are reproduced in Table 3.

Table 3

Requirements for uniformity of concrete

Test	Requirement, expressed as maximum permissible difference between results of tests of samples taken from two locations in concrete batch
Slump	
If average slump is 80mm or less	25 mm
If average slump exceeds 80 mm	40 mm
Air content, percent by volume of concrete	1.0
Coarse-aggregate content, portion by mass of each sample retained on 4.75mm test sieve percentage	6.0

In the concreting process, placing and compaction operations are most important and therefore demand careful attention and proper inspection. Proper checks during these operations will not only prevent segregation of concrete but will also avoid displacement of forms and reinforcement, secure a good bond between layers, minimise shrinkage cracking and result in a structure with a good surface finish. While compacting with mechanical vibrators, it is necessary to check that vibrations are not carried out for prolonged periods at any one position because they often lead to honeycombing and lack of homogeneity in concrete.

3.0 Conclusions

Dimensional variations are inherent in all construction processes and are related to the level of workmanship and control exercised on site. Fine tolerances require a higher level of workmanship, regular inspection schedules and tighter controls, while a lower level of workmanship will result in higher tolerances.

To achieve a high standard of workmanship, the supervisor and his staff should be familiar with acceptable dimensional tolerances for different types of work in general, and specifically with any tolerances the designer has specified for a particular construction. It is recommended that dimensional tolerance limits are clearly stated either on the construction drawings or in the specifications.

Accuracy in shape, size and alignment of concrete elements depends on the accuracy to which the formwork is built and assembled on site. Small errors in the assembly of formwork have been seen to lead to catastrophic failures.

The quality of workmanship in the placing of reinforcement bars in a concrete member is judged from the uniformity of spacing between the bars and the care exercised in securing the bars in position.

Finally, to have an acceptable quality of construction, it is essential that dimensional checks and workmanship control measures are inspected in all stages of constructional activity, including the fabrication of formwork, placing of reinforcement, batching , mixing placing, compaction and curing of concrete.

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Mathematical Modeling and Prediction Method of Concrete Carbonation

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Summary

This carbonation process of concrete is principally a diffusion phenomenon. The penetration rate of carbon dioxide depends mainly on the concrete quality and the exposure condition. Based on both the Fick first and second laws of linear diffusion equations, the three-dimensional equation of conservation of mass is derived. This equation can be reduced to two- and one-dimensional equations of conservation of mass for which provide to predict the carbonation depth beneath the corner and the general surface of concrete structures. The result of present study indicates that the maximum carbonation depth of concrete at corner is large than $\sqrt{2}$ times that of general surface under the conditions of homogeneous, isotropic and noncrack concrete. The carbonation depth predicted by the statistical method have to use large data and should be multiplied the modified coefficient for obtaining the approximate result.

1. Introduction

The reinforcing steel bars embedded in concrete are protected from corrosion by a thin oxide film that forms on their surface due to the highly alkaline, with pH values above 12.5, environment of the surrounding concrete. This alkalinity is due to calcium hydroxide ($Ca(OH)_2$) produced during the reaction between water and the constituents of cement which occurs the hardening and development of strength of cement and concrete. Carbon dioxide (CO_2) in the air penetrates concrete and reduces the pH value to less than 11, rendering it conducive to the corrosion of embedded reinforcing steel bars which cause concrete to spall or split. This process is called carbonation and is principally a diffusion phenomenon and the rate of penetration of CO_2 depends mainly on the concrete quality and the exposure condition.

The objectives of this paper are to determine the carbonation depth at the surface and the corner of a concrete member and to predict the carbonation depth by using a statistical method. In order to investigate the concrete carbonation problem, the three-dimendional equation of carbonation of mass based on the Fick second law can be reduced to a one-dimensional diffusion equation of which the solution is simplified as an empirical formula. This empirical formula associated with the statistical method can predict the carbonation depth of the concrete.

2. Mathematical modeling of concrete carbonation

Assume that concrete is a kind of homogeneous and isotropic material and is free of crack. According to Fick's first law and applying the concept of mass conservation, the mass flux per unit volume (Fig. 1) is written in the three-dimensional equation of conservation of mass as

$$\frac{\partial C}{\partial t}+\frac{\partial \eta_{CO_{2,x}}}{\partial x}+\frac{\partial \eta_{CO_{2,y}}}{\partial y}+\frac{\partial \eta_{CO_{2,z}}}{\partial z}=\gamma_{CO_2} \tag{1}$$

where C is the CO_2 mass of unit volume concrete, i. e., CO_2concentration,. $\eta_{CO_{2,x}}$ $\eta_{CO_{2,y}}$ ˋ $\eta_{CO_{2,z}}$ the mass flux of CO_2 in the x-, y-, z-direction, respectively, t time and γ_{CO_2} the absorbed CO_2 mass per unit volume per unit time or the absorbed CO_2 velocity per unit volume. Eq. (1) can be expressed in terms of vector form

$$\frac{\partial C}{\partial t}+\nabla\cdot\eta_{CO_2}=\gamma_{CO_2} \tag{2}$$

Where ∇ the Laplacian operator.

The reduction of Eq. (2) into two-dimensional case (see Fig. 2) is

$$\frac{\partial C}{\partial t}=D\nabla^2 C+\gamma_{CO_2} \tag{3}$$

where $\eta_{CO_{2,x}}=-D\frac{\partial C}{\partial x}$, $\eta_{CO_{2,y}}=-D\frac{\partial C}{\partial y}$ and D is the diffusion coefficient of CO_2 in the concrete. Fig. 3 show that the cross-sectional area of concrete located in the air can be divided into carbonated, carbonation reaction and uncarbonated zones. In the carbonated zone, the reduction of concrete absorbed CO_2 has been finished, i. e., $\gamma_{CO_2}=0$ in Eq. (3). Thus, Eq. (3) changes to

$$\frac{\partial C}{\partial t}=D\nabla^2 C \tag{4}$$

Eq. (4) is called the Fick second law of linear diffusion equation.

Assume that the ability of concrete absorbed CO_2 in unit volume is m_0 (kg/m^3). Using the

Fick first law, the carbonation front in reaction zone at any time t is

$$m_0 dx_0 dy_0 = -D\left[dy_0 \frac{\partial C}{\partial x}\Big|_{(x_0,y_0)} + dx_0 \frac{\partial C}{\partial y}\Big|_{(x_0,y_0)}\right]dt \tag{5}$$

Consider the carbonation front x_{j0} at $x_0 = y_0$ at any time t. We know

$$\frac{\partial C}{\partial x}\Big|_{(x_0=y_0)} = \frac{\partial C}{\partial y}\Big|_{(x_0=y_0)} = -\frac{C_0}{x_0} = -\frac{C_0}{y_0} \tag{6}$$

where C_0 is CO_2 concentration at the outer edge of cross section of concrete structures.

The substitution of Eq. (6) into Eq. (5) yields

$$m_0 dx_0 = 2D\frac{C_0}{x_0}dt \text{ or } m_0 dy_0 = 2D\frac{C_0}{y_0}dt \tag{7}$$

with initial condition $x_0 = 0$ and $y_0 = 0$ when $t = 0$ (Chiu, 1995). We obtain the solution of Eq. (7)

$$x_{j0} = 2\sqrt{\frac{DC_0}{m_0}t} = B\sqrt{t} \tag{8}$$

where $B = 2\sqrt{\frac{DC_0}{m_0}}$ is the coefficient of carbonation rate at the corner of the concrete structure.

Eq. (5) can be reduced into a one-dimensional problem as shown in Fig. 4 and can be written as

$$m_0 dx_0 = -D\frac{\partial C}{\partial x_0}\Big|_{x_0} dt \tag{9}$$

Consider the depth of carbonation front at time t. We know

$$\frac{\partial C}{\partial x_0}\Big|_{x_0} = -\frac{C_0}{x_0} \tag{10}$$

Substituting Eq. (10) into Eq. (9) and considering the initial condition (Tsaour, 1989), we have

$$\frac{dx_0}{dt} = \frac{DC_0}{m_0}\frac{1}{x_0}, \quad x_0 = 0 \text{ when } t = 0 \tag{11}$$

We obtain the solution of Eq. (11)

$$x_0 = \sqrt{\frac{2DC_0}{m_0}t} = A\sqrt{t} \tag{12}$$

where $A = \sqrt{\frac{2DC_0}{m_0}}$ is the coefficient of carbonation rate under the general surface of concrete structures.

Comparing Eq. (8) with Eq. (12), we get

$$B = \sqrt{2}A \tag{13}$$

Eq. (13) denotes that the maximum carbonation depth of concrete structures at corner is large than $\sqrt{2}$ times that of the general surface of concrete structures.

3. The prediction of concrete carbonation by using statistical method

Under the condition control, we take n number of A_i samples and draw the frequency distribution diagram. Based on the frequency distribution diagram, the model of statistical analysis is provided and is verified by the chi-square test or the Kolmogorov-Smirnov test (Ang and Tang, 1975). The value of A_i is determined by the carbonation depth x_i of the ith measurment point, i.e., $A_i = x_i / \sqrt{t}$. Chiu (1995) studied the distribution of coefficient of carbonation rate carbonation coefficient A and pointed out that A obeyed the normal distribution $N(\mu,\sigma)$, where μ is mean value and σ is standard deviation. Thus, the one-dimensional carbonation depth of concrete structure is $x_0(t)$, $t \in [0,T_s]$, where T_s is a determined time. The normal probability density function at any time t is

$$f(x_0,t) = \frac{1}{\sqrt{2\pi t}\sigma_A} \exp\left[-\frac{(x_0 - \mu_A\sqrt{t})^2}{2t\sigma_A^2}\right] \tag{14}$$

where $\mu_A\sqrt{t}$ is mean value and $\sqrt{t}\sigma_A$ is standard deviation. $f(x_0,t)$ determines the possible result of x_0 at any time t. According to a certain insurement rate P, the constant of β corresponding to P is determined from the table of standard normal probability. The depth of concrete carbonation beneath the general surface after service life t years is

$$x_{p|t} = (\mu_A + \beta\sigma_A)\sqrt{t} \tag{15}$$

Similarly, we can derive the depth of concrete carbonation at the corner after service life t years is

$$x_{p'|t} = (\mu_A + \beta\sigma_A)\sqrt{2t} \tag{16}$$

4. Application

The coefficient of carbonation rate A of a bridge in Taipei city with water-cement 0.45 and concrete compressive strength $f_c' = 45MPa$ obeyed the normal distribution $N(\mu_A, \sigma_A) = N(17.32, 5.1)$ (Fig. 5) and the concrete carbonation depth is of insurement rate 95%

(i. e., $p = 0.95$, $\beta = 1.645$ obtained from the table of standard normal probability) after using 50 years in service. Then, we obtain from Eq. (15)

$$x_{0.95|50} = (17.32 + 1.645 \times 5.1)\sqrt{50} = 181.79(mm)$$

This means that the carbonation depth under general concrete surface is 181.79 mm after service life 50 years. Similarly, the carbonation depth at the concrete corner after using 50 years can be estimated from Eq. (16) as

$$x_{0.95|50} = (17.32 + 1.645 \times 5.1)\sqrt{2 \times 50} = 257.09(mm)$$

Both carbonation depths mentioned above are obvious too large. This may be due to that only the 12 test samples bored from bridge are used for statistical prediction.

5. Conclusion

The carbonation depth is an important index for estimating both the damage and durability of concrete structures. This paper has presented the mathematical modeling and prediction methods of the carbonation depth for the concrete structures without any crack. It should be noted that the present study provides the maximum carbonation depth of concrete structures at corner is large than $\sqrt{2}$ times that of the general surface of concrete structures. In addition, the carbonation depth predicted by the statistical method must use large data and may be multiplied the modified coefficient for obtaining the approximate result.

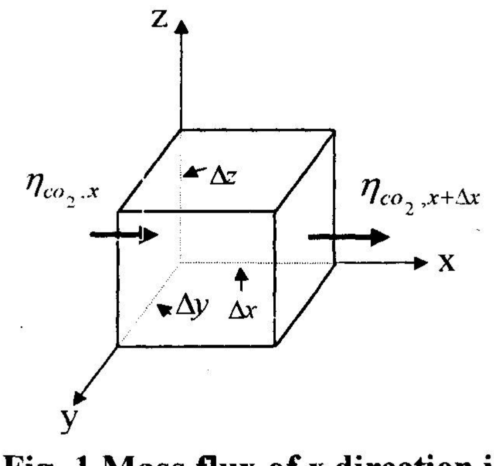

Fig. 1 Mass flux of x direction in a three-dimensional control element

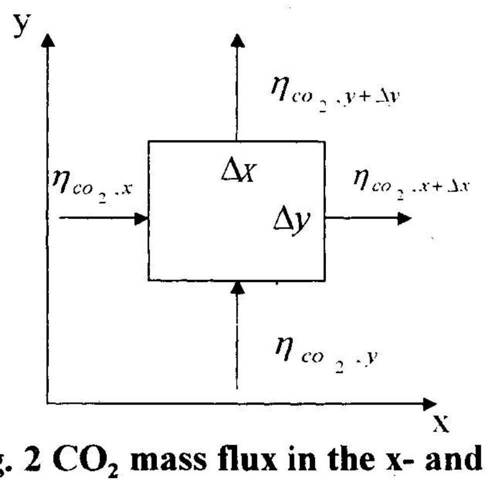

Fig. 2 CO_2 mass flux in the x- and y-direction

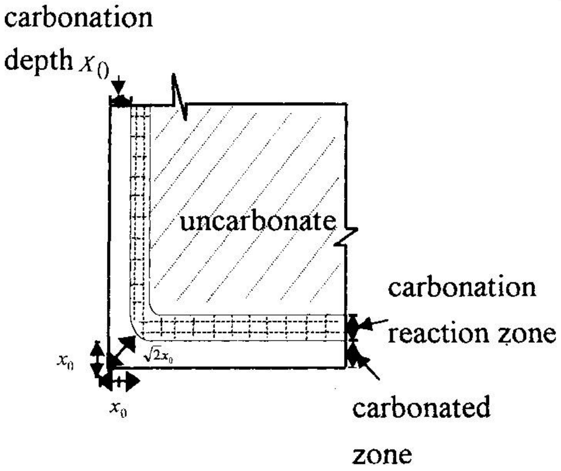

Fig. 3 Carbonation distinguish of concrete

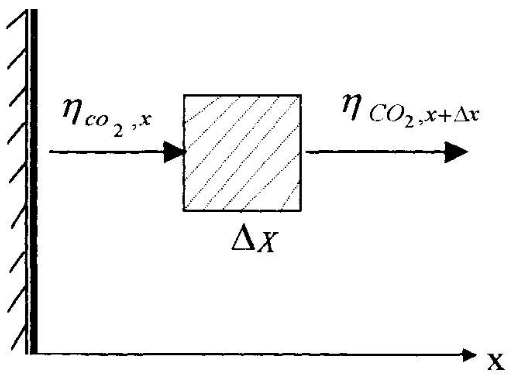

Fig. 4 CO_2 mass flux in the x-direction

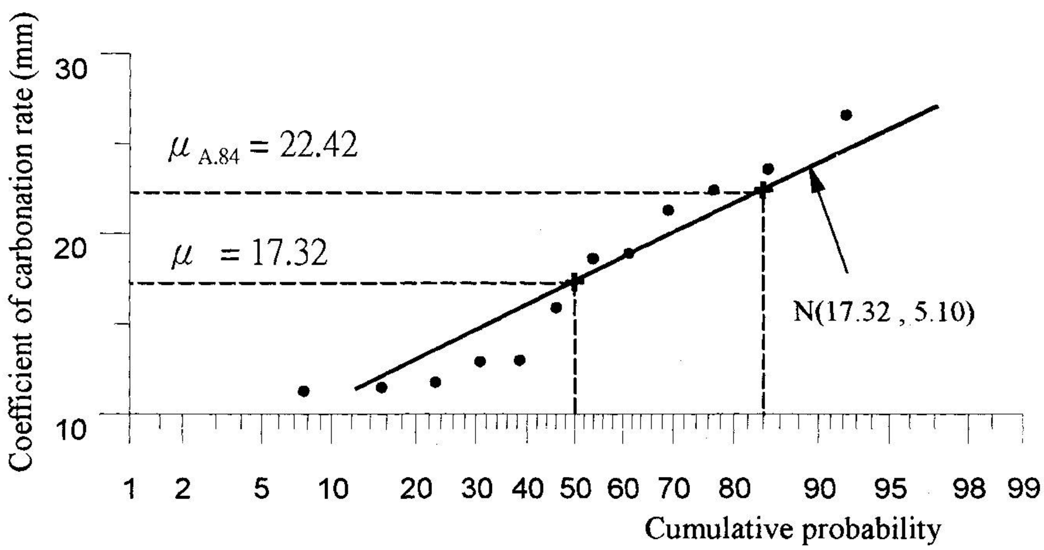

Fig 5 Coefficient of carbonation rate plotted on normal probability paper

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Deterioration Model of Concrete Bridge Girder in Urban Environment

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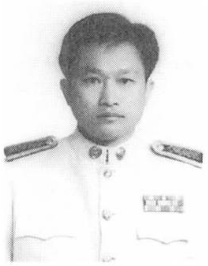

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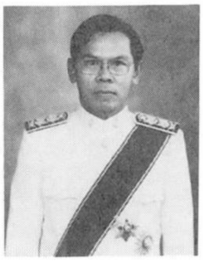

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Summary

Concrete is classified as durable materials compared to other building materials. Permeability characteristics and tensile strength are fundamentaly affect to the concrete durable properties. Aggressive anions and gas can penetrate to the concrete pores by diffusion process. This leads to the deterioration process beneath the concrete surface by physical, chemical or both actions. Concrete can also be damaged by physical process such as frost attack, abrasion, and etc. Strength, serviceability, or structural performance of concrete structures can be reduced with time by these deterioration mechanisms and processes. Such deterioration process can be predicted by mathematical function described the loss of material properties and performance with time by considering the environmental factors and concrete properties. The deterioration models can be utilized in durability design or service life prediction of concrete structures that are the useful information to impose the maintenance policies. This paper presents the basic concept of degradation modeling and mathematical models of deterioration of concrete bridge girders in Bangkok area considering the environmental factors and concrete properties.

1. Introduction

Concrete structures are distinguished as very long service life compared to other building materials. With proper service conditions concrete structures can last a definite time without reduction load - carrying capacity. This arise from the fact that the strength of concrete increases with time. Since concrete is porous material, the most important factors which influence the poor durable properties of concrete are pore size and its distribution. The durable properties of concrete are also affected by the environmental factors such as relative humidity, temperature, penetration of gasses or ions and etc. Good quality of concrete, small volume of pore and low permeability can be achieved by good curing, adequate cement content, and low water cement ratio. In aggressive environment concrete structures can be attacked by degradation factors such as mechanical, biological, chemical, and physical process. These processes can be simulated by appropriated mathematical models, which describe the degradation with time of concrete structures by considering the environmental factors and concrete properties. The purpose of this paper is to present the basic concept of degradation or performance modeling. The mathematical model of deterioration of concrete bridge girders in Bangkok area considering the environmental factors and concrete properties are used as examples.

2. Degradation Factors and Process

Degradation of concrete structures can be represented by mean degradation curve as shown in Fig 1. The curve is increased by degradation factors divided into mechanical, biological, chemical, and physical. Mathematical presentations which shown an increase of degradation with time and with appropriate parameters are called degradation models. Degradation can alternatively be presented as decrease in performance as shown by Fig 2. Mathematical presentations of structural performance with time and appropriate parameters are called performance models. Both empirical and analytical views are considered to develop the model. In general, degradation model can be proceed as the following equation.

$$\mu(D) = D(x_1, x_2, \ldots, x_n, t) \qquad \text{................ (1)}$$

where D = degradation
$\mu(D)$ = mean degradation
$x_1, x_2, \ldots, x_n$ = material ,structural ,and environmental parameters
t = the age of structure years

Performance models derived from degradation models, can also be written in the same manner.

3. Deterioration model of concrete bridge girder.

The durability models which express deterioration of concrete with time has been proposed to obtain modeling of frost attack, surface deterioration, abrasion of concrete , corrosion of reinforcement etc. This model can be used to predict the status of performance or degradation of concrete structures. The selection of relevant durability model are based on concrete properties and conditions surrounding the structures. Since concrete is porous material, so permeability is the most important durable property of concrete. Adequate cement content, low water cement ratio, and good curing can reduce capillary pore volume and permeability. The environmental conditions such as relative humidity, temperature and concentration of aggressive anions or gasses are effected to the transportation of moisture and ions within the pores in concrete. In this paper corrosion model which is the important degradation process for concrete in urban environment is considered.

Model of corrosion of steel in concrete

Steel in concrete are protected by thin oxide layer called passive film which can be formed under the condition of high alkalinity of concrete surrounding steels. The formation of passive film is called "passivation". Depassivation of steels embedded in concrete occur when aggressive anions such as chloride which penetrate to the concrete pores reach the critical concentration. Passive film can also be destroyed by the reduction of alkalinity caused by reaction of calcium hydroxide in cement paste and carbon dioxide dissolved in pore water. This is called carbonation. Corrosion of steels in concrete start after the depassivation of steel caused by these phenomenon. The rate of corrosion depend on moisture and diffusion coefficient of oxygen in concrete pores. The volume of corrosion products is many times that of the original metal. The greater need for volume causes tensile stress in concrete around the steel bar, leading to cracking or spalling of concrete cover. Corrosion model was proposed by Tutti as shown in Fig 3. This model is used for predicting the service life of steel embed in concrete. Initiation period is the time at which concentration of chloride reach the critical value or carbonation front reach steel-concrete interface. The high rate of corrosion commence after the initiation period. For conservative, initiation period is considered as the service life of structural members. In this paper carbonation induced corrosion model is considered.

Carbonation-induced corrosion

Nipon[1] has been developed model of carbonation induced corrosion for bridge structures located in urban environment .Base on Fick's first law the following expression can be derived for the depth of carbonation

$$x = \left(\frac{2D_c(C_1 - C_2)t}{a}\right)^{\frac{1}{2}} \qquad \text{................(2)}$$

where x = carbonation depth (m),
a = amount of alkaline substance in concrete,
D_c = effective diffusivity for CO_2 at a given moisture distribution in the pores ($_s$)
$C_1 - C_2$ = concentration difference of CO_2 between air and the carbonation front ($_{m^3}$)
t = time

Concrete is porous materials so the evaluation of effective diffusivity of carbon dioxide is very complex since many factors such as water cement ratio , temperature, and relative humidity are involved. Capillary pore volume is directly affected by water cement ratio while the degree of pore saturation within concrete pore is affected by relative humidity. Effective diffusivity can be obtained by the equation proposed by Papadakis et al.[2,3,4]

$$\sqrt{D_c} = B_o \varepsilon_p \left(1 - \frac{RH}{100}\right) \qquad \text{...............(3)}$$

where D_c = effective diffusivity of carbon dioxide sec^2
ε_p = cement paste porosity
B_0 = 1.2×10^{-3}
RH = relative humidity (%)

ε_p can be estimated by the following equation base on experimental data performed by Papadakis et al.

$$\varepsilon_p = 0.63w - 0.05 \qquad \text{................(4)}$$

where w = water cement ratio

The data of relative humidity and carbon dioxide concentration in Bangkok were collected. By using the appropriate confidence level the mean value of involving parameter are used in the calculation of deterioration model. Concrete strength and carbonation depth were collected from 8 bridges. Field concrete strength at the present time can be converted to 28 days strength by method proposed by CEB Model Code 1990. Water cement ratio used in equation (4) can be estimated using the relation between 28 days compressive strength and water cement ratio proposed by ACI committee 211.

material with carbon dioxide in gaseous phase. By substituting the above data to equation (2), (3) and (4) the appropriate carbonation prediction model for bridge structures can be obtained as shown by equation (5).

$$x = 13.58(0.63w - 0.05)\sqrt{wt} \qquad \text{..............(5)}$$

where x = carbonation depth (mm)
w = water cement ratio
t = time (year)

The model is verified by the actual inspection of carbonation depth as shown in Fig 4. The comparison between this model and the other models proposed by previous researchers were shown in Fig 5.

Corrosion of steel embedded in concrete

The degree of corrosion in steel bar at any time was estimated using the Faraday's law of electrolysis. This law states that the mass of any substance lost by a current is proportional to the quantity of electricity which has passed. In the mathematical form this law can be expressed by the following equation [5].

$$m = EIt \qquad \text{..............(6)}$$

where m = mass lost
I = current density
t = time
E = constant

The constant E can be obtained by the consumption of oxygen in cathodic area. Current density can be obtained using the relation of concrete pore saturation and current density proposed by Gonzalez et al [6]. Using the information of local relative humidity, which can be converted to degree of concrete pores saturation, corrosion rate are obtained for initiation and corrosion period as follow.

$$R_{ci} = 11623\ i_{ci} \qquad \text{................ (7)}$$
$$R_{cc} = 11623\ i_{cc} \qquad \text{................ (8)}$$

where R_{ci} = corrosion rate for initiation period (mm/year)
R_{cc} = corrosion rate for corrosion period (mm/year)
i_{ci} = current density for initiation period (amp/cm^2) = 1.0×10^{-4} amp/cm^2
i_{cc} = current density for corrosion period (amp/cm^2) = 6.11×10^{-7} amp/cm^2

The reduction in steel area can be calculated at the certain time using equation (7) or (8). These are useful information for calculation of the reduction of strength and stiffness of reinforced concrete members with time. The corrosion rate of 10 mm. diameter steel embed in concrete for various relative humidity was shown in Fig 6.

Mayta [7] has been studied the reduction of strength and serviceability of reinforced concrete bridge girder using the degradation model proposed by Nipon. Kasatsuek bridge which is reinforced concrete bridge located in Bangkok was used as the case study. High rate of corrosion accelerated by crack are also considered in the calculation of these performance properties. Section properties of reinforced concrete girders such as concrete area, moment of inertia and steel reinforcement area are reduced with time by carbonation and reinforcement corrosion. Using ACI method, nominal moment and shear capacity of the girder at the expected time can be calculated. Maximum deflection at mid span can be obtained by the method proposed by ACI using the concept of effective moment of inertia and mechanics of materials. Only reduction of nominal moment capacity of bridge girder was shown in Fig 7. It can be found that performance, strength and stiffness, of 65 year-old bridge girders are deteriorated with time. The higher rate of deterioration is found ,if the bridge is carrying the heavy truck HS-20.

5. Conclusion

Permeability characteristic and tensile strength of concrete play the important role of its deterioration. Degradation of concrete can be caused by mechanical, physical, chemical and the combination of these degradation processes. Degradation model of each process can be obtained by analytical or empirical method. Both of these methods are considered to develop the degradation models and then the performance models can be derived forword. The study of deterioration rate by considering the environmental factors and concrete properties of 65 year-old bridge located in Bangkok is used as illustrative example. Carbonation induced corrosion model proposed by Nipon is used as degradation model while the performance model proposed by Mayta which express the reduction of strength and stiffness of girders is used as performance model.

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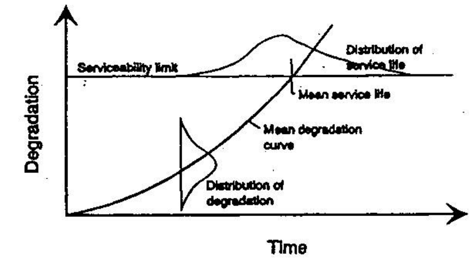

Figure 1 Degradation of concrete structures [8]

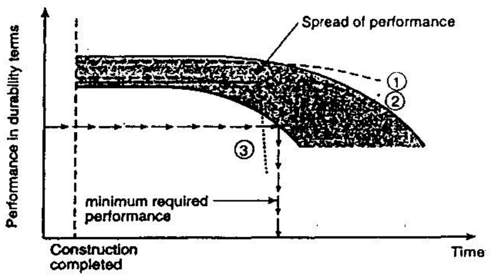

Figure 2 Performance of concrete structures [9]

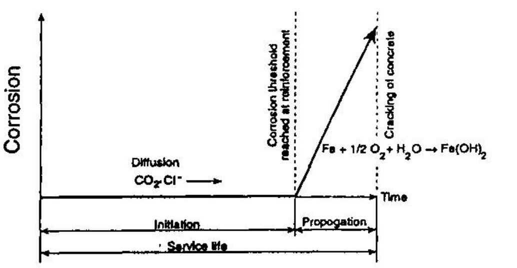

Figure 3 Corrosion model proposed by Tutti [9]

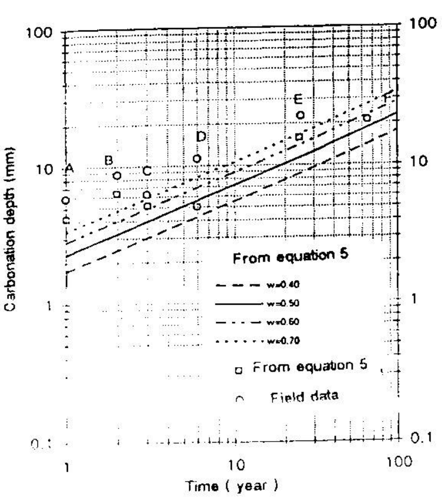

Figure 4 Comparison of carbonation depth with field data

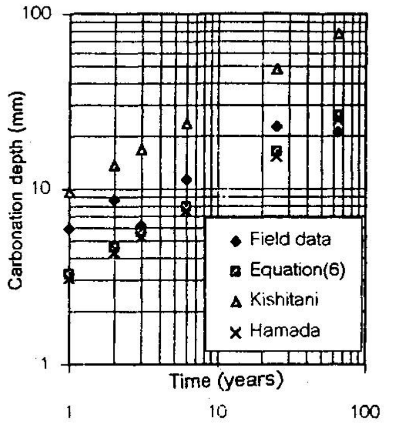

Figure 5 Comparison of carbonation depth predicted by equation (5) and other researchers, (based on average water cement ratio from field data)

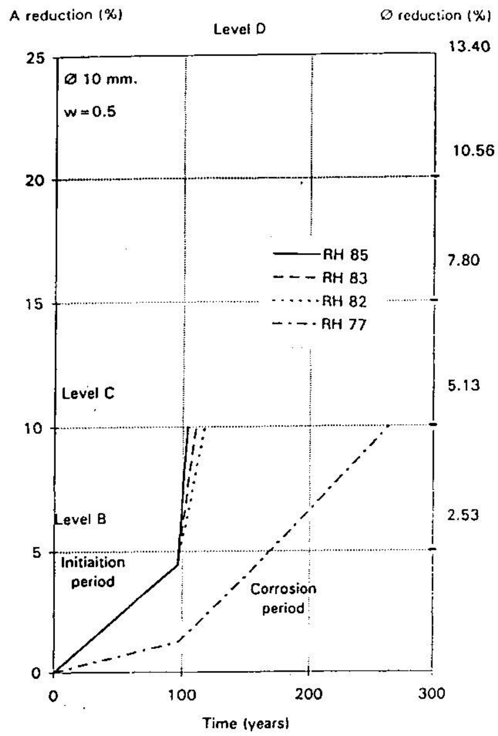

Figure 6 Corrosion rate of 10 mm. diameter steel embedded in concrete for various relative humidity.

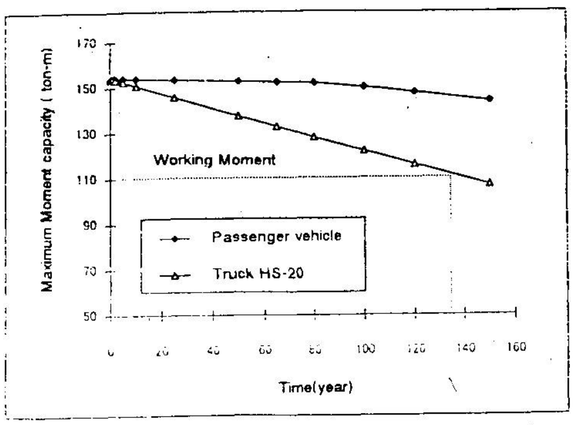

Figure 7 Reduction of moment carrying capacity of Kasatsuek bridge girder.